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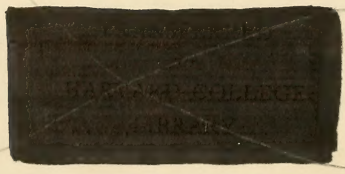

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NATIONAL ASSOCIATION

OF

CEMENT USERS

PROCEEDINGS

OF THE

SEVENTH ANNUAL CONVENTION

Held at New York, N. Y.,

December 12, 13, 14, 15, 16, 17, 19, 20, 1910

VOLUME VII

EDITED BY THE PRESIDENT

PUBLISHED BY THE ASSOCIATION
1911

513233

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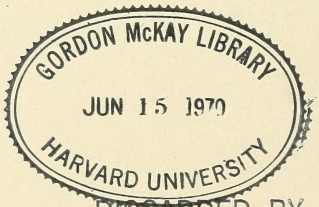

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LIST OF OFFICERS

OF

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1911.

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SECOND VICE-PRESIDENT.

ARTHUR N. TALBOT

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FOURTH VICE-PRESIDENT.

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EDWARD E. KRAUSS.

TREASURER.

HENRY C. TURNER.

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(See page 7.)

SECTIONAL COMMITTEES.

(The Chairmen are Vice-Presidents of the Association.)

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P. S. Hudson, *Chairman.*

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P. H. ATWOOD,
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ARTHUR N. TALBOT,
SANFORD E. THOMPSON.

ROADWAYS, SIDEWALKS AND FLOORS.

C. W. Boynton, *Chairman.*

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J. B. LANDFIELD,
C. R. MILLER,
A. E. SNODGRASS.

SPECIFICATIONS AND METHODS OF TESTS FOR CONCRETE MATERIALS.

Sanford E. Thompson, *Chairman.*

CLOYD M. CHAPMAN,
WILLIAM B. FULLER,
RUSSELL S. GREENMAN,
ARTHUR N. TALBOT.

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ALFRED HOPKINS, H. H. QUIMBY.

EDUCATION.

LOGAN WALLER PAGE, *Chairman.*

PERCY H. WILSON, *Secretary.*

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WILLIAM G. HARTRANFT. A. MOYER.
W. A. HOLMAN. J. P. H. PERRY.

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Arkansas—MARTIN NELSON.
California—LEROY ANDERSON.
Colorado—ALVIN KEYSER.
Connecticut—CHARLES A. WHEELER.
Delaware—HARRY HAYWARD.
Florida—J. J. VERNON.
Georgia—LEROY C. HART.
Idaho—W. L. CARLYLE.
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Iowa—J. B. DAVIDSON.
Kansas—W. W. JARDINE.
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Louisiana—W. R. DODSON.
Maine—GEORGE E. SIMMONS.
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Minnesota—JOHN T. STEWART.
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New York—H. W. RILEY.
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Wisconsin—CHARLES A. OCOCK.
Wyoming—J. C. FITTERER.

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President.	1905	JOHN P. GIVEN. (Presiding Officer First Convention.
	1905–10	RICHARD L. HUMPHREY.
First Vice-President.	1905	A. L. GOETZMANN.
	1906–9	MERRILL WATSON.
	1909–10	EDWARD D. BOYER.
Second Vice-President.	1905–6	JOHN H. FELLOWS.
	1907–10	M. S. DANIELS.
Third Vice-President.	1905	H. C. QUINN.
	1906–7	O. U. MIRACLE.
	1908	S. B. NEWBERRY.
	1909–10	E. S. LARNED.
Fourth Vice-President.	1905–7	A. MONSTED.
	1908–9	GEORGE C. WALTERS.
	1909–10	F. A. NORRIS.
Treasurer.	1905	A. S. J. GAMMON.
	1906–10	H. C. TURNER.
Secretary.	1905–6	CHARLES C. BROWN.
	1907	W. W. CURTIS.
	1908–9	GEORGE C. WRIGHT.

CHARTER

OF

THE NATIONAL ASSOCIATION OF CEMENT USERS.

KNOW ALL MEN BY THESE PRESENTS, That we, the undersigned, all of whom are citizens of the United States, and a majority of whom are residents of the District of Columbia, have associated ourselves together for the purpose hereinafter set forth and desiring that we may be incorporated as an Association under sub-chapter three (3) of the Incorporation Laws of the District of Columbia, as provided in the Code of Law of the District of Columbia, enacted by Congress and approved by the President of the United States, do hereby certify:

1. Name. The name of the proposed corporation is "The National Association of Cement Users."

2. Term of Existence. The existence of the said corporation shall be perpetual.

3. Objects. The particular business and objects of the said corporation shall be to disseminate information and experience upon and to promote the best methods to be employed in the various uses of cement by means of convention, the reading and discussion of papers upon materials of a cement nature and their uses, by social and friendly intercourse at such conventions, the exhibition and study of materials, machinery and methods and to circulate among its members by means of publications the information thus obtained.

4. Incorporators. The number of its managers for the first year shall be fifteen.

In Witness Whereof, we have hereunto set our hands and seals this fourteenth day of December, A. D. 1906.

RICHARD L. HUMPHREY, (SEAL)
JOHN STEPHEN SEWELL, (SEAL)
S. S. VOORHEES. (SEAL)

OFFICE OF RECORDER OF DEEDS,
DISTRICT OF COLUMBIA.

This is to certify that the foregoing is a true and verified copy of a Certificate of Incorporation, and of the whole of such Certificate as received for record in this office at 9:49 A. M., the 19th day of December, A. D. 1906.

In testimony whereof I have hereunto set my hand and affixed the seal of this office, this 20th day of December, A. D. 1906.

(Signed) R. W. DUTTON,
Deputy Recorder of Deeds,
District of Columbia.

(SEAL)

BY-LAWS

OF

THE NATIONAL ASSOCIATION OF CEMENT USERS.

Article I.

MEMBERS.

Section 1. Any company or persons engaged in the construction or maintenance of work in which cement is used, or qualified by business relations or practical experience to co-operate in the purposes of this Association, or engaged in the manufacture or sale of machinery or supplies for cement users, or a man who has attained eminence in the field of engineering, architecture or applied science, is eligible for membership.

Sec. 2. A company shall be treated as a single member and allowed but one vote.

Sec. 3. Any member contributing annually twenty or more dollars in addition to the regular dues shall be designated and listed as a Contributing Member.

Sec. 4. Application for membership shall be made to the Secretary on a form prescribed by the Executive Board. The Secretary shall submit monthly or oftener if necessary to each member of the Executive Board for letter ballot a list of all applicants for membership on hand at that time with a statement of the qualifications, and a two-thirds majority of the members of the Executive Board shall be necessary to an election.

Applicants for membership shall be qualified upon notification of election by the Secretary by the payment of the annual dues, and unless these dues are paid within 60 days thereafter the election shall become void. An extract of the By-Laws relating to dues shall accompany the notice of election.

SEC. 5. Resignations from membership must be presented in writing to the Secretary within thirty days after the close of the fiscal year and shall be acceptable provided the dues are paid for that year.

ARTICLE II.

OFFICERS.

SECTION 1. The officers shall be the President, the Vice-Presidents, the Secretary and the Treasurer, who, together with the five latest living Past-Presidents, shall constitute the Executive Board. Vacancies occurring during the year shall be filled by the Executive Board.

SEC. 2. The Elective Members of the Executive Board consisting of the President, the First, the Second, the Third and the Fourth Vice-Presidents, shall be elected annually by ballot at the convention at a business session fixed by the Executive Board and shall hold office until their successors shall qualify.

SEC. 3. The Elective Members of the Executive Board shall appoint the Secretary and the Treasurer; they shall create such special committees as may be deemed desirable for the purpose of preparing recommended standards concerning the proper use of cement for consideration by the Association, and shall appoint a chairman for each committee who shall be a Vice-President of the Association. Four additional members on each special committee shall be appointed by the President, in consultation with the Chairman.

SEC. 4. It shall be the duty of the Executive Board to audit the accounts of the Secretary and the Treasurer before each annual convention.

SEC. 5. The Executive Board shall appoint a Committee on Nomination of Officers and a Committee on Resolutions, to be announced by the President at the first regular session of the annual convention.

SEC. 6. There shall be an Executive Committee of the Executive Board consisting of the President, the Secretary, the Treasurer and two of its members, appointed by the Executive Board.

SEC. 7. The Executive Committee shall manage the affairs

of the Association during the interim between the meetings of the Executive Board.

Sec. 8. The President shall have general supervision of the affairs of the Association. He shall preside at the Annual Convention, at the meetings of the Executive Board and the Executive Committee, and shall be ex-officio member of all committees.

The Vice-Presidents in order of seniority shall discharge the duties of the President in his absence.

Sec. 9. The Secretary shall perform such duties and furnish such bond as may be determined by the Executive Board.

Sec. 10. The Treasurer shall be the custodian of the funds of the Association and shall disburse the same in the manner prescribed by the Executive Board. He shall furnish bond in such sum as the Executive Board may determine.

Sec. 11. The Secretary and the Treasurer shall receive such salaries as may be fixed by the Executive Board.

Article III.

MEETINGS.

Section 1. The Association shall meet annually. The time and place shall be fixed by the Executive Board and notice of this action shall be mailed to all members at least thirty days previous to the date of the Convention.

Sec. 2. The Executive Board shall meet during the Convention at which it was elected, effect organization and transact such business as may be necessary.

Sec. 3. The Executive Board shall meet at least twice each year. The time and place to be fixed by the Executive Committee.

Article IV.

DUES.

Section 1. The fiscal year shall commence on the first of July and all dues shall be payable in advance.

Sec. 2. The annual dues of each member shall be five dollars ($5.00).

Sec. 3. A member whose dues remain unpaid for a period of one year shall forfeit the privilege of membership and shall be officially notified to this effect by the Secretary, and if these dues are not paid within thirty days thereafter his name shall be stricken from the list of members. Members may be reinstated upon the payment of all charges upon the books of the Association.

Article V.

STANDARD SPECIFICATIONS.

Section 1. Proposed Standard Specifications to be submitted to the Association must be mailed to the members at least thirty days prior to the Annual Convention, and as there amended and approved, passed to letter ballot, which shall be canvassed within sixty days thereafter, such specifications shall be considered adopted unless at least ten per cent. of the total membership shall vote in the negative.

Article VI.

AMENDMENT.

Section 1. Amendments to these By-Laws, signed by at least three members, must be presented in writing to the Executive Board prior to November 1st and shall be printed in the notice of the annual convention. These amendments may be discussed and amended at the annual convention and passed to letter ballot by a two-thirds vote of those present. Two-thirds of the votes cast by letter ballot shall be necessary for their adoption.

SUMMARY OF THE PROCEEDINGS OF THE SEVENTH ANNUAL CONVENTION.

MONDAY, DECEMBER 12, 1910, 3.00 P. M.

Meeting of Section on Roadways, Sidewalks and Floors.

President Humphrey in the Chair.

This meeting was devoted to a general discussion covering the preparation of materials, laying, finishing, prevention of dusting and costs.

FIRST SESSION—MONDAY, DECEMBER 12, 1910, 8.00 P. M.

The Convention was called to order by President Richard L. Humphrey.

The President announced that the Oesterreichischer Beton Verein had sent two delegates to present the felicitations of that Society and presented a cablegram received from the Deutscher Beton Verein extending cordial greetings and best wishes for the success of the Convention; also a cable from the Swiss Association for Testing Materials and a letter from the Concrete Institute of England transmitting greetings.

Mr. George McAneny, President of the Borough of Manhattan, delivered an address of welcome to the City of New York on behalf of the Mayor, the Honorable William J. Gaynor, as follows:

Mr. George McAneny:

Mr. President and Gentlemen: I am very fortunate in being permitted, in the absence of the Mayor, to receive you on behalf of the City of New York. You are, I imagine, an advance guard of many hundreds who are coming here to discuss your own plans, your own work and the wonderful developments that are gradually taking form in our age of concrete.

To the City of New York itself, to its people and to its own fraternities of engineers, contractors and others of your craft, your coming is a matter of signal importance. It means that your products

are to be brought right to our doors; that opportunity is to be given to listen to what you have to say; and that we are also to have the opportunity to witness in physical demonstration the actual development of the age of concrete, and the wonderful and multifarious uses to which your arts have been applied with wonderful results.

To those of us who are in the city service and who have certain problems within our care, certain responsibilities connected with the solution of these problems, this appeals particularly. I feel my own opportunity to come here to this opening session the more, because it so happens that in the peculiar form of government of this town, while the Mayor is Mayor of all the people and has the great centralized departments under his control, the wise or unwise dispensation of the Charter, making providence some time ago, gave over to the Borough Presidents, the Presidents of the five boroughs into which the city is divided, the care of all the public works, that is, the streets, the highways, the sewers and everything that lies either upon or under them; the control of private building operations, the custody of the public buildings, responsibility for their design, to a large degree, and for their construction and maintenance. Roughly speaking, everything that one sees out of doors in New York, is in the care of the Borough Presidents. Under their supervision comes a corps of engineers, architects and others who have to do directly with these things. Your problems are brought very closely home to this branch of the administration of the City of New York.

I rejoice to see that the Superintendent of Buildings in my own Borough, my own appointee, for whose discovery and selection I take a great amount, I think a pardonable amount of pride, Rudolph P. Miller, is a Sectional Vice-President and Chairman of one of your Committees. In our Highway Department, George W. Tillson, who I think it will be conceded is among the first of those who have given their lives to engineering and highway problems, will be my delegate to your conference, and the city at large will send Nelson P. Lewis, Chief Engineer of the Board of Estimate and Apportionment.

Gentlemen, I doubt whether a large proportion of the people of this or any other town realize to what a great degree their future, that is, the future of the cities in which they live, is dependent upon the things that are entrusted to your care, the things that you have taken over. I doubt whether very many have followed the wonderful progress which has been made in this branch of engineering, or at least the branches of engineering, that teach the magic uses of cement. I confidently look towards that time when you will deftly turn out in two or three days, not merely modern houses built for our occupancy, but most of the ordinary things that we find within those houses. I trust that you will stop at our daily food. I presume that may be a reasonable limit to place upon the progress I see before me.

In the upbuilding of the cities your part is going to be a greater

and greater one. Is there any line of public work in which you do not enter and in which you do not hold a controlling part? If there is, I have not yet found or observed it. You are brought inevitably into the lives of the cities themselves. You are the builders. Yours is your own responsibility; yours will be the greater satisfaction of the future.

When I think, for instance, of the wonderful thing that New York has grown to be, of this aggregation of nigh on towards six millions of people within our own geographical limits and within the limits of the two New Jersey counties that belong to the great metropolitan district—when I think of the work that still lies ahead in making this city what it must be in the future, I realize that we have been too careless in permitting it to grow up with so little regard for engineering plan. I hope to see, in the near future, the time when the City of New York will have the service of the greatest minds that it can bring to the task of the working out of a city plan that looks far, far ahead, that takes those territories that now lie undeveloped or practically so, but where the statisticians and seers tell us in due time the center of the city will lie, and turns them into the city of the future, that will embrace our subways, our boulevards and parks, our public buildings and every item of our commercial development, with a long look towards the future.

Over in Paris, when a new avenue is opened up to-day, though some old, and so far as a greater part of the city is concerned, very possibly some forgotten slum, when something new of any sort goes on the map of Paris, in all likelihood it was conceived years ago. The plan of Baron Haussman, unchanged to-day, is still working out as it were. In our own Washington they are even to-day working out the plan of streets and boulevards and grouping of public buildings as conceived in the mind of Major L'Enfant more than a hundred years ago. And still New York has no plan; still New York grows very largely by chance. I dare say it is true of almost every city of the country. When we do take up the upbuilding of a greater and finer city, the imperial New York of the future, as we proceed through all that work, in whatever direction we turn, you will find us depending upon you, upon the arts and the crafts of the profession you represent. Upon your brains and your vigor and that of men like yourselves must depend in great measure, in short, the physical future of the cities of America.

And, Gentlemen, let me again congratulate our town upon your presence here, because you are bound to do us good. Let me wish you in the name of the City a thoroughly successful conference. May you find us as agreeable as you have ever hoped to find us. The City is yours; do with it as you like. You haven't time now to rebuild it, but in the course of the next week or ten days at least take from it all the enjoyment it can give you and believe that you are more than doubly welcome.

President Humphrey responded to this welcome:

I am sure of voicing the will of the Association in thanking Mr. McAneny for the cordial welcome he has extended to the City of New York and for the inspiring picture he has painted of our future work. We hope that our conference here will prove beneficial and that we may profit by the many fine examples of concrete work being executed and that we may in turn give to those who help to make this convention a success, information that will repay them for their efforts. In coming to this City the Association is actually the gainer as it will enable it to grow. In New York City there have been so many big things done in concrete and reinforced concrete, the inspection of which has been arranged by our Committee on Excursion and it is hoped that as many members as possible will find time to make the inspection of the work to which we have been so cordially invited.

The Association is very fortunate in having as one of its speakers a man who stands as a type of the eminent engineers of this country, a man who has done big things and of whom still bigger things are expected. It is with great pleasure that I introduce to you Mr. John A. Bensel, President of the American Society of Civil Engineers and New York State Engineer Elect, who will address you on behalf of the engineering interests of the City.

Mr. John A. Bensel:

Mr. President and Gentlemen: A convention like yours brings to one forcibly the great strides that are made in engineering lines, particularly in your case in the adaptation of concrete and cement to the uses and the purposes of mankind. When I first began to take an interest in things engineering all of the important harbor work, in which I was engaged at that time in this City, was fabricated, so far as the structure was concerned, from imported cement; and it was only in a comparatively—in one sense—recent time, about twenty years ago, that we found people calculating on the importation of volcanic rock and even a further adulteration by grinding with a good brand of cement a certain proportion of sand to bring the cement to a reasonable price so it could be used.

All of this is not a thing of the very remote past, but within a period of between twenty and twenty-five years ago. It may be on account of the great advance you have made, irrespective of the years, that you may have a certain amount of satisfaction in what has been accomplished, and think that you may have a certain venerable distinction. Here, however, it might be well for you to remember the saying that to consider well the proportion of things and resolve that, "It is better to be a young June bug than an old bird of paradise." I used to be able to best realize the strides of recent years in engineering

affairs, by meeting my old friend Charles Haswell, and thinking of the things he had seen in his practice. Now that he is gone there would seem to be nothing better to show the advancement made than to consider cement in its adaptation to the world's uses, as showing how fast the world is moving.

The President:

Mr. Bensel has offered new views on a subject which is of greatest interest to us all and I feel sure that we will carefully consider the suggestions made. On behalf of the members I wish to express their appreciation and thanks to Mr. Bensel.

The Address of Welcome on behalf of the Building Trades of New York will be made by Mr. Ross F. Tucker in the absence, through illness, of Mr. Benjamin D. Traitel, President of the Building Trades Employers' Association, and I take great pleasure in introducing to you Mr. Ross F. Tucker.

Mr. Ross F. Tucker:

Mr. President and Gentlemen: It is my very pleasant mission to extend to you the greetings of the Building Trade Employers' Association; and while I deeply regret that Mr. Traitel is unable to be present and while he undoubtedly would speak to you much more gracefully, I am sure he could not do so more sincerely.

My identification with the cement industry has extended over a long period. I have seen the growth, during my active career, of the production of American Portland cement from 800,000 to more than 70,000,000 barrels a year. I have seen cement adapted to almost every engineering and architectural problem. But a new application arises in my mind to-night and that is, to cement the friendship between the National Association of Cement Users and the Building Trades Employers' Association.

A few words as to what the Association is may not be amiss. It consists of some eleven hundred firms or corporations in the City of New York, representing an invested capital perhaps of $20,000,000, banded together in thirty-three trade associations, of which The Masters' League of Cement Workers is one, with the purpose primarily of regulating labor conditions. It has nothing whatever to do with prices or restrictions of trade or the commercial side of buildings, but it has a great deal to do with maintaining industrial peace, and which problem has been very largely worked out by the Building Trades Employers' Association through a plan of arbitration that went into existence in 1903 and is to-day in use. Through that plan of arbitration I presume 90 per cent. of all the controversies arising between employers and employed are settled, so that the Association stands

before the community with a practical guarantee that you can pull down a building say on the corner of Broadway and Twenty-sixth Street on the first day of May and have a 20-story skyscraper there on the first day of the next May without interference from trade unions. It takes the stand that the men have all the rights of organization that the employers have, but it denies the proposition that "might makes right." It maintains that where a proper or a just controversy arises between employer and employed there is no issue that reasonable, sensible, quiet-minded men cannot settle across a table without resort to strikes, intimidation or force.

That is the general purpose of the Building Trades Employers' Association, and it has done a great work for the building industry in this city and for the public at large. We have quarters on Thirty-second Street, known as the Builders' Exchange. On the second floor is an exhibition room of building materials of various kinds and their many applications. Some time ago the Association offered to architects prizes for the design of a $2,500 and a $4,500 house in competition, and a large number of drawings were sent in; they are now on exhibition and are presently to be judged by a committee of well-known architects in this city.

On the top floor, the twelfth, are the Association rooms, and it is my pleasant privilege on behalf of the Association to extend to you the courtesies of the Building Trades Employers' Association and to ask you to please go there and make yourselves at home. The club is open at all times and the members will be very glad to see the members of the National Association of Cement Users at any time they please to come.

The President then responded:

I am quite sure the members will accept the hospitality that has been extended by Mr. Tucker and the invitation to become better acquainted with the organization he represents. I hope as many of you as possible will avail yourselves of the facilities offered. On behalf of the Association, therefore, I desire to thank Mr. Tucker in appearing here to-night to extend greetings on behalf of the building interests of the City.

The Official Delegates and foreign guests were formally presented to the officers and members of the Association.

The President announced the appointment of the following committees by the Executive Board:

Committee on Nomination of Officers:

Thomas M. Vinton, *Chairman*, New York, N. Y.
Robert Anderson, Cincinnati, Ohio.

A. T. Bradley, Rochester, N. Y.
Robert A. Cummings, Pittsburgh, Pa.
Emile G. Perrot, Philadelphia, Pa.

Committee on Resolutions:

B. F. Affleck, *Chairman*, Chicago, Ill.
Peter Gillespie, Toronto, Canada.
A. L. Johnson, St. Louis, Mo.
Angus B. MacMillan, Boston, Mass.
Milton Dana Morrill, Washington, D. C.

The following papers were read and discussed:

"Dustless Concrete Floors," L. C. Wason.

"Some New Methods in Sidewalk and Curb and Gutter Construction," Jerome B. Landfield.

The report of the Committee on Roadways, Sidewalks and Floors, C. W. Boynton, Chairman, was then presented.

The meeting adjourned until Tuesday at 10.30 A. M.

TUESDAY, DECEMBER 13, 1910, 9.00 A. M.

Meeting of the Section on Concrete and Reinforced Concrete.

President Humphrey opened the meeting and turned the Chair over to A. E. Lindau, Chairman of the Committee on Reinforced Concrete and Building Laws.

The meeting took the form of a general discussion on Design, Methods of Construction, Selection of Materials, Costs, etc.

SECOND SESSION—TUESDAY, DECEMBER 13, 1910, 10.30 A. M.

President Humphrey in the Chair.

The following papers were presented and discussed:

"A Test of a Flat Slab Floor in a Reinforced Concrete Building," Arthur R. Lord.

"A Discussion of the Basis of Design for Reinforced Concrete Floor Slabs," Arthur R. Lord.

The meeting adjourned until 3.00 P. M.

THIRD SESSION—TUESDAY, DECEMBER 13, 1910, 3.00 P. M.

President Humphrey in the Chair.

A paper by Emile G. Perrot on "Analysis of Results of Load Tests on Panels of Reinforced Concrete Buildings" was, in the absence of the author, read by W. A. Ballinger. This was followed by a discussion.

In the absence of the author, the paper on "The Calculation of Reinforced Concrete Plates Supported by Four Columns," by L. J. Mensch, was read by title.

Robert A. Cummings presented a paper on "Some Methods of Measurement of Concrete Construction."

The report of the Committee on Reinforced Concrete and Building Laws was presented by the Chairman, Alfred E. Lindau.

The following papers presented by the Hungarian Society of Architects and Engineers were read by title as their translation had not been completed:

"Regulations Relative to Design and Construction in Reinforced Concrete."

"Resolutions Relative to the Uniform Designation of the Binding Materials used in the Preparation of Concrete and other Resolutions Governing the Uniform Transportation and Examination of Cements."

A paper on "The Human Element in Concrete Construction," by Harry F. Porter, was, in the absence of the author, read by title.

The meeting adjourned until 8.00 P. M.

FOURTH SESSION—TUESDAY, DECEMBER 13, 1910, 8.00 P. M.

President Richard L. Humphrey in the Chair.

Richard L. Humphrey presented the Annual Address of the President on "Some Fallacies in Methods of Fireproofing."

An address on "The National Fire Protection Association and Its Work" was then presented by Wm. H. Merrill.

The following papers were read and discussed:

"Some Thermal Properties of Concrete," Charles L. Norton.

"Some Special Uses of Concrete in Mining," George S. Rice.

The meeting adjourned until Wednesday at 10.30 A. M.

Wednesday, December 14, 1910, 9.00 a. m.

Meeting of the Sections on Fireproofing and on Insurance.

President Richard L. Humphrey opened the meeting and called William H. Ham, Sectional Vice-President, to the Chair.

The meeting took the form of a general discussion on the construction of reinforced concrete buildings from an insurance point of view and the effect on insurance rates.

Fifth Session—Wednesday, December 14, 1910, 10.30 a. m.

President Richard L. Humphrey in the Chair.

In the absence of the author, James MacDonald, the paper on "Moving Forms for Reinforced Concrete Storage Bins" was read by title.

The following papers were read and discussed:

"Construction Problems in Concrete Bridges," W. N. Denman.

"The Effect of Electrolysis on Metal Imbedded in Concrete," Cloyd M. Chapman.

In the absence of the chairman, R. P. Miller, the report of the Committee on Fireproofing was read by title.

The report of the Committee on Insurance was then presented by the Chairman, William H. Ham. The Committee offered the following resolution, which was unanimously approved:

Whereas, The Standards of this Association include specifications for forms of concrete construction which if followed will improve the state of this art, reduce the loss of life and property by fire and generally improve the Standards of Building Construction in America; be it

Resolved, That these facts and these Standard Specifications referred to be called to the attention of the State and Municipal Building Departments and the various organizations of Fire Underwriters in the United States, accompanied with a request that these Standards receive recognition in existing laws and in schedules for estimating insurance rates.

On motion, a vote of thanks was extended to the Committee on Insurance for its services.

Mr. Raymond E. Brooks, Secretary of the Concrete Products Manufacturers' Association of New Jersey, presented a brief statement of the aims and purposes of that organization.

Business Session.—The report of the Executive Board and the minutes of the meetings of the Board were approved as read. A resolution presented by the Board was referred to the Committee on Resolutions.

The Executive Board presented the following amendment to the By-Laws, which was approved and ordered to letter ballot:

Amend Article I, Section 4, so that the second sentence reads as follows:

The Secretary shall submit monthly, or oftener if necessary, to each member of the Executive Board for letter ballot a list of all applicants for membership on hand at that time with a statement of their qualifications, and a two-thirds majority of the members of the Executive Board shall be necessary to an election.

Amend Article I, Section 4, by adding the following paragraph:

Applicants for membership shall be qualified upon notification of election by the Secretary by the payment of the annual dues, and unless these dues are paid within 60 days thereafter the election shall become void. An extract of the By-Laws relating to dues shall accompany the notice of election.

On motion, the appointment of a Committee on Specifications and Methods of Test for Concrete Materials was referred to the Executive Board.

The Committee on Nomination of Officers, Thomas M. Vinton, Chairman, reported as follows:

President, Richard L. Humphrey, Philadelphia, Pa.
First Vice-President, Edward D. Boyer, Catasauqua, Pa.
Second Vice-President, Arthur N. Talbot, Urbana, Ill.
Third Vice-President, E. S. Larned, Boston, Mass.
Fourth Vice-President, Ira H. Woolson, New York, N. Y.

On motion, duly seconded, the report of the Committee was accepted and the nominees declared elected.

On motion, the following resolution was unanimously adopted:

Resolved, That the Association extend to M. S. Daniels a vote of thanks as a small token of appreciation and recognition of his able and faithful services as Second Vice-President for the long term of six years.

The meeting adjourned until 8.00 P. M.

Sixth Session—Wednesday, December 14, 1910, 8.00 P. M.

President Richard L. Humphrey in the Chair.

A paper on "Problems Encountered in the Construction of the Aziscohos Storage Dam" was presented by Seth A. Moulton.

The following paper was read:

"The Preparation and Handling of Concrete," H. M. Cryder.

"The Spouting of Concrete," by F. E. Engstrum, read by title in the absence of the author.

George L. Prentiss then presented a paper on "The Use of Compressed Air in the Handling of Mortar and Concrete," followed by a demonstration.

The meeting adjourned until Thursday, 10.30 A. M.

Thursday, December 15, 1910, 9.00 A. M.

Meeting of the Section on Specifications for Cement Products.

President Richard L. Humphrey opened the meeting and called C. K. Arp, member of the Committee on Specifications for Cement Products, to the Chair.

Discussion on the manufacturing, curing, cost and other matters pertaining to Cement Products.

Seventh Session—Thursday, December 15, 1910, 10.30 A. M.

President Richard L. Humphrey in the Chair.

Charles D. Watson presented a paper on "General Considerations in the Construction of a Cement Products Plant" which was followed by discussion.

The following papers were presented and discussed:

"An Incident of the Value of Concrete in Reducing the Cost of Insurance," Emile G. Perrot.

"Cement Tile Plant—Layout and Operation," C. M. Powell.

The report of the Committee on Specifications for Cement Products was presented by C. K. Arp, embodying:

(*a*) Proposed Standard Specifications for Architectural Concrete Blocks.

(*b*) Proposed Standard Specifications for Plain Concrete Drain Tile.

After discussion the proposed Specifications were referred back to the Committee for revision and presentation at the next Convention.

F. S. Phipps then presented a paper on "Additional Notes on Steam Curing Plants."

The meeting adjourned until Friday at 10.30 A. M.

THURSDAY, DECEMBER 15, 1910, 8.00 P. M.

The first Banquet of the Association was held at the Hotel Astor, about 400 members and guests being in attendance.

The President, Richard L. Humphrey, opened the Banquet with appropriate remarks, introducing the Toastmaster, Robert W. Lesley.

The following toasts were responded to:

"New York City and Concrete," John Purroy Mitchell.

"International Technical Co-Operation," Karl Bittner.

"Concrete and the Better Housing of the Poor," Robert W. DeForrest.

"The Architect and Concrete," John M. Carrere.

"The Building Interests of the City," Benjamin D. Traitel.

"A Concrete Talk," Charles Battell Loomis.

"Address," John R. Morron.

Friday, December 16, 1910, 9.00 a. m.

Meeting of the Section on Treatment of Concrete Surfaces.

President Richard L. Humphrey opened the meeting and called upon H. H. Quimby, member of the Committee on Treatment of Concrete Surfaces, to preside in the absence of the Chairman.

The meeting took the form of a topical discussion on the artistic treatment of concrete surfaces.

Eighth Session—Friday, December 16, 1910, 10.30 a. m.

President Richard L. Humphrey in the Chair.

In the absence of the author, Peter Gillespie, the paper on "Notes on Web Reinforcement" was read by title.

The paper on "The Web Reinforcement of Concrete Beams," by John Stephen Sewell, was, in the absence of the author, read by the President.

The following papers were read and discussed:

"Comparative Cost and Maintenance of Various Types of Building Construction," J. P. H. Perry.

"Concrete Filled Steel Arches," H. H. Quimby.

In the absence of the author, the paper on "Reinforced Concrete Sewers," by J. A. Hooke, was read by title.

The meeting adjourned until 3.00 p. m.

Ninth Session—Friday, December 16, 1910, 3.00 p. m.

President Richard L. Humphrey in the Chair.

L. C. Wason, Chairman, presented the report of the Committee on "Treatment of Concrete Surfaces," embodying the following:

(*a*) Proposed Specifications for Portland Cement Stucco.

(*b*) Proposed Specifications for Scrubbed Concrete Surfaces.

The report on Treatment of Concrete Surfaces was accepted as read.

The Proposed Specifications for Cement Stucco were referred back to the Committee for revision and with instructions to report at the next Convention a specification covering all forms of Stucco.

The proposed Specification for Scrubbed Concrete Surface was ordered to letter ballot.

On motion, the question of the appointment of a committee on Nomenclature was referred to the Executive Board.

The meeting adjourned until 8.00 P. M.

TENTH SESSION—FRIDAY, DECEMBER 16, 1910, 8.00 P. M.

President Richard L. Humphrey in the Chair.

A paper on "The Henry Hudson Memorial Bridge" was presented by William H. Burr.

In the absence of the author, a paper on "Concrete Construction; Testimony of the Roman Forum and Some Modern Examples," by Alfred Hopkins, was read by title.

The following papers were read:

"The Use of Reinforced Concrete for Hospitals and Similar Structures," R. A. McColloch.

"Reinforced Concrete School Buildings," John T. Simpson.

The meeting adjourned until Saturday at 9.00 A. M.

ELEVENTH SESSION—SATURDAY, DECEMBER 17, 1910, 9.00 A. M.

President Richard L. Humphrey in the Chair.

In the absence of the author, the paper on "Tufa Concrete," by Joseph B. Lippincott, was read by the President.

An abstract of the "Specifications for the Delivery and Testing of Trass," presented by the Hungarian Society of Architects and Engineers, was read by the President.

A paper on "The Taylor Concrete Coal Breaker" was read by R. D. Richardson.

The following papers were read and discussed:

"The Use of Reinforced Concrete in Sea Water," R. Baffrey.

"The Relation of Lime Content of Cement to the Durability of Concrete," Henry S. Spackman.

"Waterproofing with Water," Cloyd M. Chapman.

In the absence of the author, a paper on "The Insulation of Concrete Structures" by Edward W. DeKnight was read by title.

A. H. Harrison presented a paper on "The Waterproofing of Tunnels."

The Committee on Resolutions, B. F. Affleck, Chairman, then reported as follows:

Whereas, The Oesterreichischer Beton Verein, of Austria, has shown its interest in the work and has honored this Association by sending two official delegates to the Seventh Annual Convention in order that there may be closer affiliation and international co-operation, be it

Resolved, That the thanks and appreciation of this Association be recorded and tendered to the Oesterreichischer Beton Verein for its interest in the work, and to the delegates, Mr. Karl Bittner and Mr. Franz Boehm, for the time and attention they have devoted to this convention.

Be it resolved, That the thanks of the Association be extended to the members of the Oesterreichischer Beton Verein, of the Deutscher Beton Verein (E.V.) and the Hennebique Construction Company for their courtesy in sending photographs illustrative of foreign concrete construction in response to the invitation extended by this Association.

Resolved, That the thanks of this Association be tendered hereby to all the officers of the organization, and that this Association acknowledge its debt of gratitude especially to its President, Mr. Richard L. Humphrey, for his untiring efforts and faithful work which have to so great an extent placed this Association to-day on a higher plane of usefulness, and with a larger and better membership than ever before attained.

Whereas, The work of this Association and the field of its activities has been greatly enlarged during the past year by dues from contributing members, and by donations from individuals and corporations interested in the work, therefore, be it

Resolved, That the Association hereby express its thanks to those who have thus generously aided in the work, and that the Secretary be directed to send a copy of this resolution with a personal letter to

each contributing member, and to each individual firm or corporation from whom donations have been received.

RESOLVED, That reports and specifications adopted by this body be brought to the attention and formally laid before city councils when considering building codes, school boards and commissioners of public buildings everywhere.

WHEREAS, The Standards of this Association include specifications for forms of concrete construction, which, if followed, will improve the state of this art, reduce loss of life and property by fire, and generally improve the standard of building construction in America, be it

Resolved, That these facts and these standard specifications referred to be called to the attention of State and Municipal Building Departments, and the various organizations of fire underwriters in the United States and Canada, with the request that these standards receive recognition in existing laws and in schedules for estimating insurance rates.

RESOLVED, That the thanks of this Association be hereby extended to the members and others who have aided, by the preparation and presentation of papers, the members and the local sectional committees, and to the technical and trade press for co-operation and assistance in furthering the interests of this Association.

On motion, the resolutions were unanimously adopted.

The President then declared the Convention adjourned *sine die*.

SATURDAY, DECEMBER 17, 1910, 8.00 P. M.

President Richard L. Humphrey in the Chair.

The Honorable Martin B. Madden delivered an illustrated address on "The Construction of the Panama Canal."

OFFICIAL DELEGATES, SEVENTH ANNUAL CONVENTION.

Foreign Representatives.

OESTERREICHISCHER BETONVEREIN.

Karl Bittner, Director, Vienna, Austria.
Franz Boehm, Engineer, Vienna, Austria.

M. Banneth, Special Representative of the Lord Mayor, Budapest, Hungary.

SOCIÉTÉ DES INGENIEURS CIVILS DE FRANCE

Joseph Boero, Neuilly, Seine, France.

United States Government.

DEPARTMENT OF AGRICULTURE.

Office of Public Roads.

Logan Waller Page, Director, Washington, D. C.

DEPARTMENT OF THE INTERIOR.

Bureau of Mines.

George S. Rice, Mining Engineer, Pittsburgh, Pa.

Reclamation Service,

Frederick H. Newell, Director, Washington, D. C.

DEPARTMENT OF THE NAVY.

Bureau of Yards and Docks,

Lieut. Frederic R. Harris, Brooklyn, N. Y.

DEPARTMENT OF THE TREASURY.

Office of Supervising Architect,

Alfred B. Fry, Superintendent of Repairs, New York, N. Y.

DEPARTMENT OF WAR.

Corps of Engineers,

COL. S. W. ROESSLER, U. S. A., New York, N. Y.
COL. WILLIAM M. BLACK, U. S. A., New York, N. Y.

Isthmian Canal.

CAPT. F. C. BOGGS, U. S. A., General Purchasing Officer, Washington, D. C.

STATES.

SOUTH DAKOTA.

SAMUEL H. LEA, State Engineer, Pierre, South Dakota.

MUNICIPALITIES.

DISTRICT OF COLUMBIA.

MAJOR W. V. JUDSON, U. S. A., Engineer Commissioner.

NEW YORK.

City at Large,

NELSON P. LEWIS, Chief Engineer, Board of Estimates and Apportionment.

Borough of Bronx.

AMOS SCHAEFFER, Consulting Engineer.

Borough of Brooklyn.

R. WALTER CREUZBAUR, Consulting Engineer.

Borough of Manhattan.

GEORGE W. TILLSON, Chief Engineer, in Charge of Highways.

Borough of Richmond.

LOUIS L. TRIBUS, Commissioner Public Works.

PHILADELPHIA.

GEORGE S. WEBSTER, Chief Engineer, Bureau of Surveys.
HENRY H. QUIMBY, Assistant Engineer of Bridges.

BOSTON.

L. K. ROURKE, Superintendent of Streets.

ST. LOUIS.

J. A. HOOKE, Assistant Sewer Commissioner.

BALTIMORE.

EDWARD D. PRESTON, Inspector of Buildings.

PITTSBURGH.

N. S. SPRAGUE, Superintendent, Bureau of Construction.

CAMDEN.

L. F. FARNHAM, City Engineer.
ALFRED SAYERS, Commissioner of Streets and Highways.

LOCAL COMMITTEE ON ARRANGEMENTS
SEVENTH ANNUAL CONVENTION

THOMAS M. VINTON, *Chairman*

J. P. H. PERRY, *Secretary*

RECEPTION

IRA H. WOOLSON, *Chairman*
R. BAFFREY
E. T. CAIRNS
OLAF HOFF
HORACE D. LOOMIS
CLIFFORD RICHARDSON
FRANK C. WIGHT

EXCURSIONS

E. P. GOODRICH, *Chairman*
ALFRED CRAVEN
GEORGE C. CLARKE
W. B. FULLER
NELSON P. LEWIS
J. L. PULTZ
J. WALDO SMITH

BANQUET

J. P. H. PERRY, *Chairman*
G. E. ESHER
E. B. LITCHFIELD
R. P. MILLER
ALBERT MOYER
B. H. RADER

PUBLICITY

P. AUSTIN TOMES, *Chairman*
W. H. BEACH
J. P. BECK
F. B. MARSH
MERRILL WATSON

REGISTRATION AND INFORMATION

R. W. BOYD, *Chairman*
T. H. BOORMAN
C. M. CHAPMAN
C. H. OTT
RONALD TAYLOR

HOTELS AND TRANSPORTATION

F. G. WEBBER, *Chairman*
E. L. RANSOME
R. F. TUCKER
M. M. UPSON

EXHIBIT OF PHOTOGRAPHS ILLUSTRATING FOREIGN CONCRETE CONSTRUCTION.

The Association extended an invitation to foreign concrete organizations to exhibit at the Seventh Annual Convention, photographs and drawings illustrative of representative work of their members. An exceedingly interesting and instructive exhibition of over 1,000 photographs and drawings was effected, the contributors being as follows:

Austria.

Oesterreichischer Betonverein.

Ast & Company, Ed., Wien.
Faccanoni, Luigi, Wien.
Gaertner, E., Wien.
Marinelli & Faccanoni, Wien.
Pittel, Adolf Baron, Wien.
Pittel & Brausewetter, Wien.
Rella & Neffe, N., Wien.
Wayss & Cie, G. A., Wien.
Wayss, Westerman & Cie, Wien.

France.

Hennebique Construction Company, Paris.

Germany.

Deutscher Betonverein.

Drenkhahn & Sudhop, Braunschweig.
Dücker & Cie, Düsseldorf.
Dyckerhoff & Widman, Dresden.
Hüser & Cie, Oberkassel.
Liebolt & Cie, Holzminden.
Rank, Gebr., München.
Weinrich & Reinken, Kiel.
Windschild & Langehott, Dresden.
Wolle, Rud., Leipzig.

National Association of Cement Users.

PROCEEDINGS

OF THE

SEVENTH CONVENTION

This Association is not responsible, as a body, for the statements and opinions advanced in its publications.

SOME FALLACIES IN METHODS OF FIREPROOFING.

Annual Address by the President,

Richard L. Humphrey.*

The story told by the great conflagrations that have swept some of the large cities of this country cannot be too often repeated because it is a lesson more effective than all lectures or warnings that can be given. It is a matter of wonder to most of those interested in fire losses as to how long the people of the United States will continue to remain indifferent to these enormous annual losses. These great conflagrations have appalled the people for brief periods, but have not been sufficient to secure the adoption of effective and efficient laws which would forever prevent their recurrence. The San Francisco conflagration in a period of three days produced losses that wiped out the profits of twenty years in the insurance business. It was recently stated by one of the prominent insurance men of this country that if a fire should destroy the City of New York south of Fourteenth street, it would bankrupt every insurance company in the country.

While the public is momentarily aroused by these great confla-

*Consulting Engineer, Philadelphia, Pa.

grations, in the absence of a well defined policy it is difficult to utilize this aroused public sentiment to secure the enactment of efficient laws. It is necessary first, that those who prepare them shall be sufficiently familiar with the requirements that there may be enacted laws which will really prove effective. The contrast between the policy pursued by the European countries and that of this country is glaring. In Europe the practice is to study the means of preventing fire and maintain a sufficient fire brigade for the purpose of putting out fires in the contents of a building. In

FIG. 1.—MILLS BUILDING, SAN FRANCISCO, AFTER FIRE.

this country, our lax building laws permit the erection of almost any type of structure and the public treasury is largely devoted to the maintenance of expensive fire fighting brigades. In the one case large sums of money are expended in securing the best fire resistive buildings possible, in the other case the money is expended in providing apparatus for putting out fires in non-fire-resistive buildings. So proud are we of our splendid and efficient fire brigades and methods of fighting fires that we are prone to drift into the comfortable feeling that our fire service is all that is necessary to afford a proper protection.

Under ordinary circumstances, such service would doubtless prove efficient, but one has but to recall the Baltimore fire where bitter winter weather so seriously crippled the water supply as to render fire fighting a rather primitive process. In the San Francisco fire, the crippling of the water mains by the earthquake

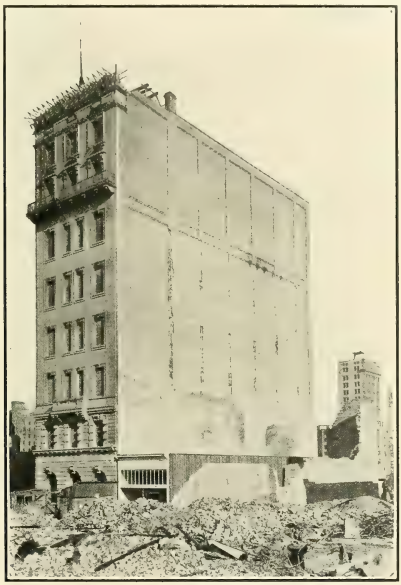

FIG. 2.—PACIFIC STATES TELEPHONE AND TELEGRAPH BUILDING, SAN FRANCISCO, AFTER FIRE.

rendered practically useless the fire fighting service; the little work done was by means of long lengths of fire hose taking water from San Francisco Bay. It has been frequently stated that some day the City of New York will have a conflagration which will destroy so much property as to paralyze business and possibly induce a panic. This statement is always ridiculed by the champions of

the fire brigade, and yet it frequently happens that fires under difficult conditions in winter are not controlled as quickly as they should be; as an instance of this, the Parker Building stands as a monument of the extent even the finest fire fighting equipment may be rendered useless through the failure of the water supply. It should not be considered for a moment that a proper fire fighting service is unnecessary, on the contrary, with adequate building laws which insure structures reasonably fire resisting, it is absolutely essential that there shall be maintained a service which can

FIG. 3.—PALACE HOTEL, SAN FRANCISCO, AFTER FIRE.

quickly extinguish any fire which might occur in the contents of such buildings. It will probably be a matter of great difficulty to make adequate laws retroactive, and we must wait for the gradual substitution of fire resistive types of buildings for many of the inflammable fire traps that are a menace to our large cities at the present time. While it may be debatable as to whether or not these laws should be retroactive in all parts of the city, it certainly is a fact that within the fire limits of our great cities no structure should be permitted which did not fulfill the requirements of a reasonably fire-resistive building regardless of its occupancy.

The unusual low per capita losses of Europe in contrast with the enormous losses of this country furnish a story which we are prone to use on all occasions for the purpose of pointing out the

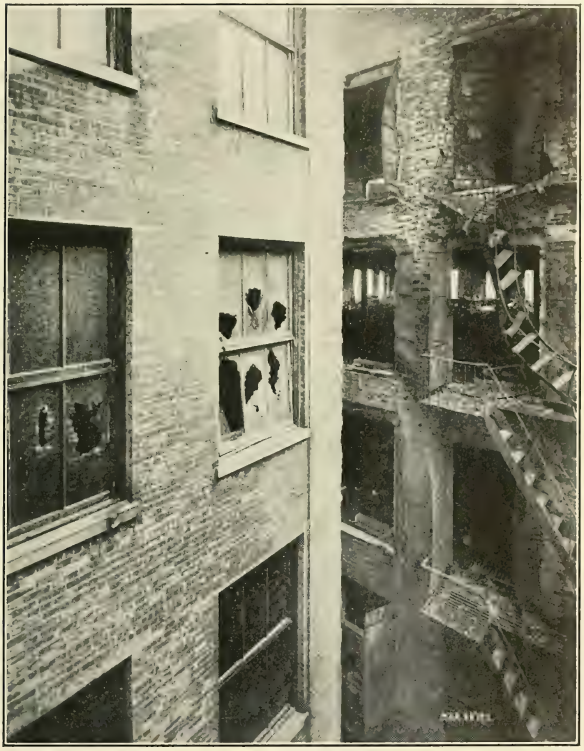

FIG. 4.—FIREPROOF BUILDING ADJACENT TO ASCH BUILDING, NEW YORK, SHOWING PROTECTION AFFORDED BY WIRE GLASS IN METAL FRAMES.

superiority of European methods compared with those in this country. Laws of all kinds are necessarily a matter of evolution and it becomes necessary to increase their strictness with the increase

in density of population, so that while it might be permissible to erect structures of wood in a small town, such structures should not be permitted in a large city, because of their menace to surrounding property. Our building laws are gradually becoming more and more strict, and we are gradually acquiring better fire-resistive types of buildings. It should be remembered that in many of the foreign countries there is an abundance of stone and slate in

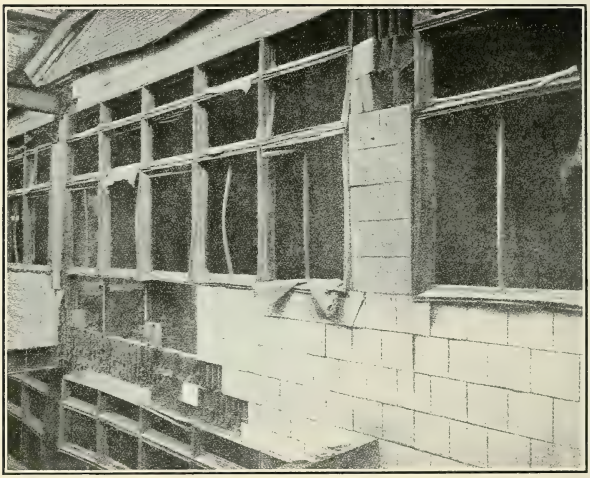

FIG. 5.—WELLS-FARGO BUILDING, SAN FRANCISCO, SHOWING EFFECT OF FIRE.

contrast to the abundance of timber in this country, and this has resulted in even dwelling houses being constructed of stone walls and slate roofs, and thereby producing a building of high fire resistance.

It would seem apparent in developing a better condition as regards the danger from fire in this country, that one of the first things is to establish the comparative value of the various materials that are used for fireproofing purposes, and to test out the various methods in use. Unfortunately, most of the data

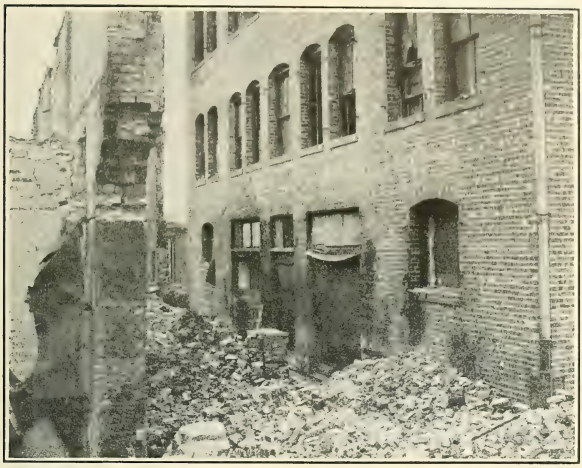

FIG. 6.—VOLKMAN BUILDING, SHOWING PROTECTION AFFORDED BY FIREPROOF DOORS AND WINDOWS.

FIG. 7.—REINFORCED CONCRETE BUILDING OF DAYTON MOTOR CAR COMPANY, WITH ADJOINING NON-FIREPROOF BUILDING DESTROYED BY FIRE.

available resulted from great conflagrations, as we have not the means of making as careful and accurate comparisons as might be possible under more controllable conditions. Even when defects are found in materials of supposedly high fire-resistive properties, it requires a great deal to correct the previous false impressions.

The object of this paper is to point out a few fallacies in the commonly employed methods of fireproofing in this country.

A study of the San Francisco conflagration reveals the fact that there were large unbroken areas of frame buildings which easily succumbed to the flames. Occasionally a building of a fire-resisting type would form at least a partial barrier or check to the sweep of the flames. A building must not only be fire-resistive, as regards its contents, but also to fires in adjoining buildings. In order to prevent the spread of the fire, dynamiting was resorted to in San Francisco; this might have proved effective if water had been available to so dampen the destroyed buildings as to render them unburnable, but in the absence of water through the failure of the water supply, the ruins accelerated, rather than retarded, the fire.

A building, however, with properly protected openings, not only checks the fire, but retards it, in that there is no fuel added to the flames. A building well designed, with properly protected openings and non-burnable walls and roof, becomes a barrier to external flames and serves as an additional barrier to the adjoining buildings in that it is possible to confine the fire within the building itself, and when the interior of the building is properly broken up with fire walls, it is possible to confine the fire to the point of origin. Many buildings are destroyed by exterior fires which gain access through insufficiently protected doors and windows.

A casual examination of the exterior of the Mills Building, San Francisco, Fig. 1, would lead one to believe that it had satisfactorily passed the conflagration. A more careful examination will show, however, as is the case with many of the other buildings of this character, that the windows, doors, ornamental trim, and a large percentage of the fireproofing were completely destroyed, and a number of the columns buckled. The salvage in this building was less than 25 per cent., which leads to the natural query, has the building successfully withstood the fire?

The Pacific States Telephone and Telegraph Building, Fig. 2, differs from the preceding structure in that it was provided with protected windows and doors and, therefore, suffered little damage, although surrounded by a fire which was probably hotter than that which destroyed the Mills Building. Unfortunately, there was a weakness in this building, an imperfect barrier in the rear doorway, through which the fire gained access and did some damage.

The famous Palace Hotel, Fig. 3, was a wooden joist building, only the brick walls of which survived, because of their substantial construction. It is sometimes pointed out that brick is an admirable fireproof material for buildings, but we must bear in mind that we cannot build beams or floors with brick, and that it can only be used for piers and walls, and while this material passed the fire admirably, in the case of the Palace Hotel, the value of the remaining brick walls was but a small percentage of the entire cost of the building. The reason bricks are so much better in their fire resistance than other clay products is that while they expand under heat quite as much as other building materials of clay, they are of small size and the relatively larger number of joints enables the expansion to take place without visible damage to the structure.

Fig. 4 shows the building adjoining the Asch Building, New York, in which the metal frame wire glass windows served as an effective barrier against this fire. The holes in the wire glass were punched by the firemen, who used these windows as shields in fighting the fire in the Asch Building. This clearly shows if a building is to be properly protected by fireproof windows, these windows must be of the proper type. The view given in Fig. 5 shows the court of the Wells-Fargo Building, in which ordinary glass was used in metal frames, the melting and failure of the glass destroyed whatever merit there may have been in the frames themselves, and the barrier was little better than an ordinary wood frame window.

Fig. 6 shows the exterior of the Volkman Building, where may be seen fireproof doors and windows of the approved type, which not only resisted the fire from without, but also successfully withstood the blows of the falling walls of the adjoining building.

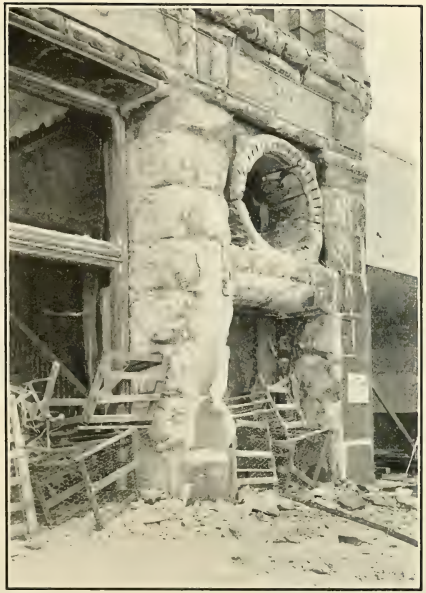

FIG. 8.—ORNAMENTAL STONE WORK DAMAGED BY FIRE, ENTRANCE TO ÆTNA BUILDING, SAN FRANCISCO.

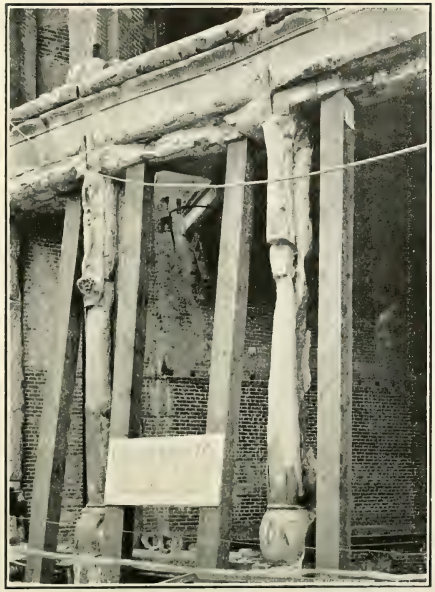

FIG. 9.—GRANITE COLUMNS DESTROYED BY FIRE, HOBART BUILDING, SAN FRANCISCO.

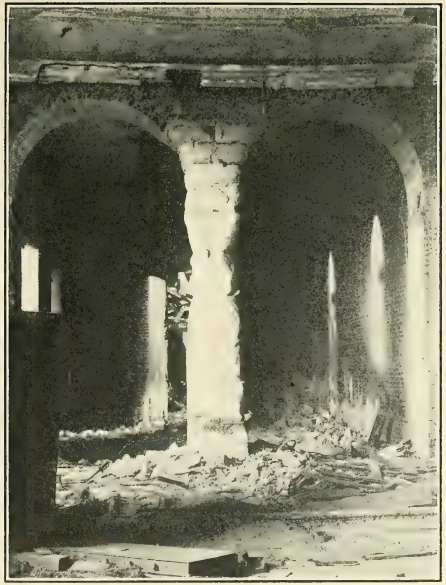

FIG. 10.—INTERIOR STONE COLUMN, CITY HALL, SAN FRANCISCO.

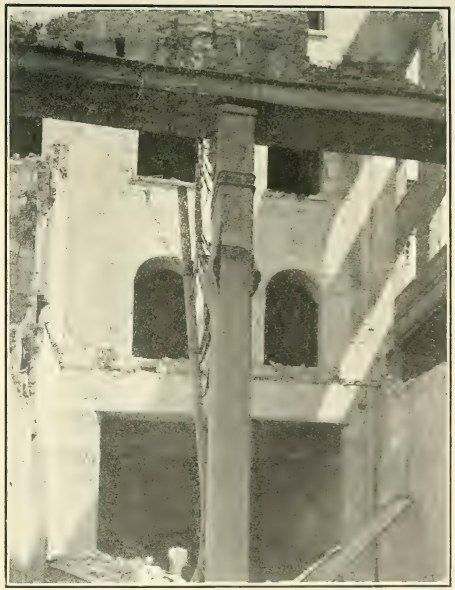

FIG. 11.—FAILURE OF CAST IRON COLUMN, LOAD CARRIED BY CONCRETE FILLING, ACADEMY OF SCIENCES BUILDING, SAN FRANCISCO.

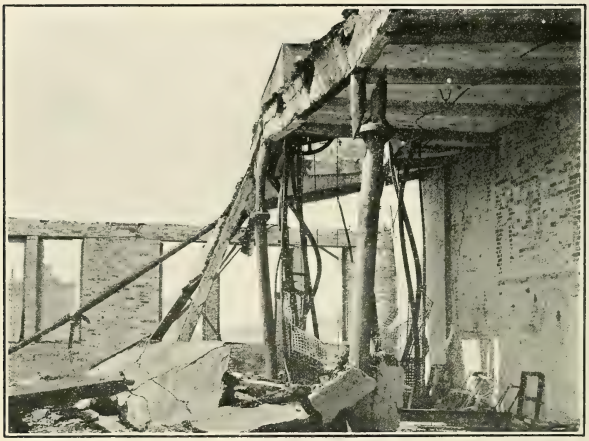

FIG. 12.—FAILURE OF CAST IRON COLUMNS, CAUSING COLLAPSE OF CINDER CONCRETE FLOOR.

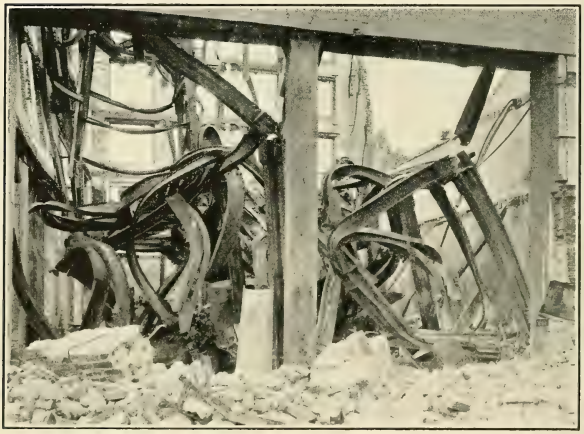

FIG. 13.—RESULT OF FIRE IN BUILDING WITH UNPROTECTED STEEL STRUCTURAL MEMBERS.

How effective barriers of this kind may be in preventing the spread of the flames is illustrated in Fig. 7, of the Dayton Motor Car Company, where it will be seen that a fire on one of the floors of the reinforced concrete building did not spread to the adjoining floors, but did spread to and completely destroyed the adjacent non-fireproof building. It is interesting to note that the cost of the repairs to the reinforced concrete building after this fire was comparatively small, while the adjoining building suffered practically a total loss.

In taking up the various phases of fire-resistive types of buildings, we should bear in mind that the trim or ornamental parts of the structure constitute a very high percentage of the total cost, and it is, therefore, desirable to use in the buildings material which will suffer the least damage from fire. The preliminary investigations of the fire-resistive properties of building materials which were conducted by the United States Geological Survey at the Underwriters' Laboratories in Chicago, showed that even our natural building stones require investigation in order to develop the best manner of quarrying for maximum fire resistance. It was found that it made considerable difference how the stone was quarried, and that a fairly high resistance might be obtained by quarrying the stone in the proper way. Many stones, such as granite, spall badly under the action of the flames.

The view of the Ætna Building entrance, Fig. 8, shows how badly the ornamental stone work was damaged by the flames. In fact, most of it was so seriously damaged as to destroy its value. Where natural stone is used as a structural column, the damage by flames becomes much more serious, as illustrated in Fig. 9, which shows two granite columns in the Hobart Building, San Francisco, that were so badly spalled by the fire as to necessitate wooden props being placed in order to prevent the structure from collapsing. Fig. 10 is an interior column of the City Hall, whose strength, as may be readily seen, was reduced more than 50 per cent. through the action of the flames. It is quite evident then that an unprotected stone column is an extremely poor structural member, the destruction occurring from rapid expansion under heat, although the stone itself is unburnable. There is as much reason, therefore, for protecting a stone column as there is

to protect a steel or a concrete column or any other structural member.

The writer understands that the cast iron columns supplied for the Academy of Sciences Building, San Francisco, Fig. 11, were too light, and in order to stiffen them sufficiently to carry the proposed loads, the interior was filled with concrete. The fire

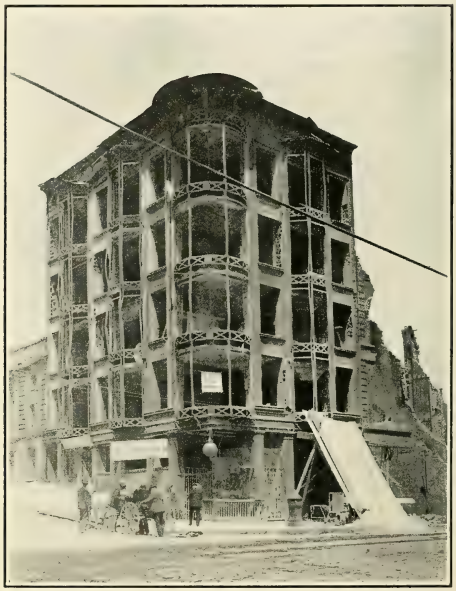

FIG. 14.—EFFECT OF FIRE ON BUILDING WITH NON-FIREPROOF EXTERIOR.

which destroyed the portion of the building supported by these columns, expanded the unprotected iron and caused it to crack, as shown, but the concrete remained uninjured and continued to carry the load.

Fig. 12 illustrates how the failure of the one member of a structure may cause a failure of the surrounding parts, and lead to a false conclusion as to the cause of the failure. In this case,

the failure was ascribed by some to the failure of the cinder concrete floor, and not to the cast iron columns. As a matter of fact, the failure of the cast iron columns in the fire withdrew the support to the cinder concrete floor and caused its collapse.

A common fallacy is the belief that because a building is constructed with walls of brick or stone, with columns, girders and

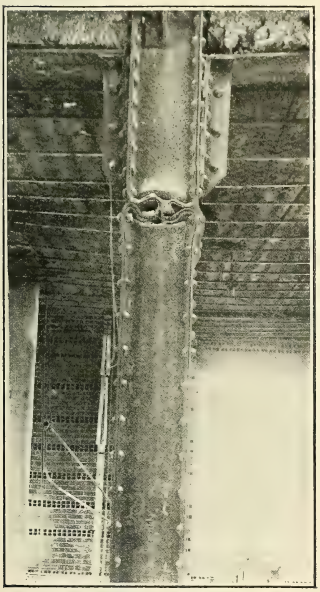

FIG. 15.—FAILURE OF COLUMNS INCASED IN METAL FABRIC COVERED WITH HARD PLASTER, FAIRMOUNT HOTEL, SAN FRANCISCO.

FIG. 16.—BUCKLED COLUMN, HOTEL HAMILTON, SAN FRANCISCO.

beams of steel and with a floor of concrete, that the building is fireproof. One has only to look at Fig. 13 to see how foolish this supposition is. Here the total collapse of the unprotected steel work of such a building stands as a monument to bad construction, and of the fallacy of calling such a structure fireproof.

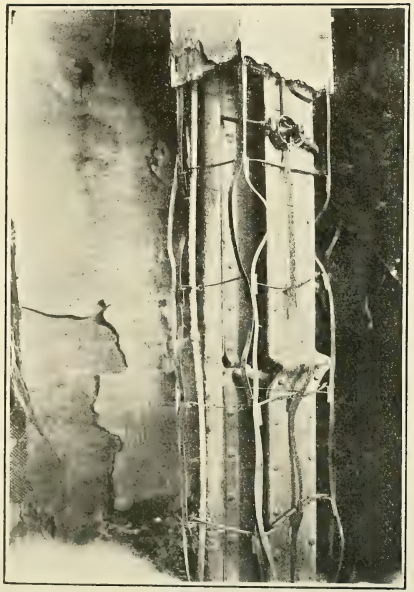

FIG. 17.—FAILURE OF COLUMN PROTECTED BY SINGLE LAYER OF WOVEN WIRE COVERED WITH HARD PLASTER.

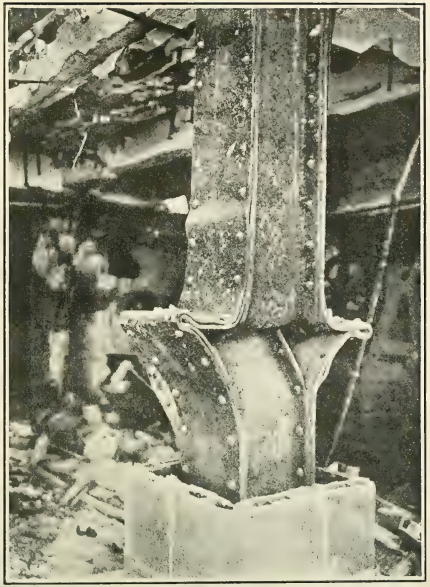

FIG. 18.—FAILURE OF STEEL COLUMN, FIREPROOFED WITH TERRA COTTA.

It often happens that the floors and columns of a building are properly constructed for reasonable fire resistance, but the building is provided with flimsy walls and window openings, through which the fire gains access to and destroys the contents. While it is true that the floors and columns are still standing in the building shown in Fig. 14, the entire exterior is so damaged that the value of what remains forms but a small part of the original cost of the building.

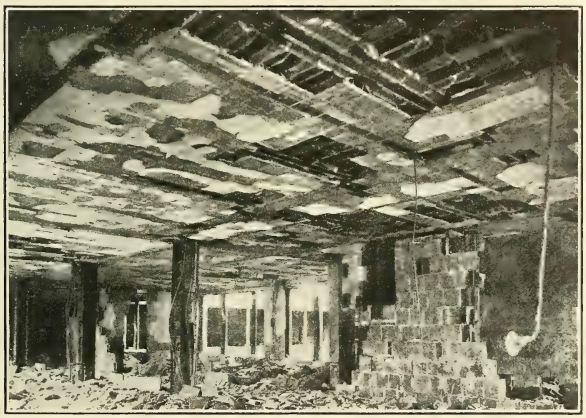

FIG. 19.—FAILURE OF STEEL COLUMN FIREPROOFED WITH TERRA COTTA, DUE TO BUCKLING OF ENCLOSED PIPES FROM EXPANSION.

There is perhaps no greater fallacy than in the matter of fireproofing steel columns. Fireproofing of the flimsiest character is placed in buildings where a large number of people congregate and, after all the decorations and trim of the building are in place, the occupants enjoy a false sense of security, in the belief that the building is fireproof. A notable example of an inefficient method of fireproofing is afforded by the Fairmount Hotel, San Francisco, where more than one hundred steel columns buckled under the heat produced by the small amount of combustible material which was found on the various floors of this

uncompleted building. This fireproofing consisted in bending the metal fabric forming the partition walls, around the columns and giving it a coat of plaster. In other words, the entire protection afforded the column consisted in perhaps ¾ in. of hard plaster. This protection proved entirely inadequate, and the columns failed in the manner illustrated by Fig. 15.

FIG. 20.—FAILURE OF STEEL COLUMN THROUGH EXPANSION OF ENCLOSED PIPES DESTROYING THE TERRA COTTA FIREPROOFING.

The view, Fig. 16, shows a buckled column in the Hotel Hamilton, San Francisco, which was protected in this manner, and it will be observed that the metal fabric, which has been removed so as to show the buckling in the column, also serves to show how flimsy this method of fire protection actually is.

In Fig. 17 may be observed a column protected with or-

dinary woven wire, covered by a single layer of hard plaster. In this case there were imbedded the various pipes and electric conduits, and the buckling of these resulted in the stripping of the plaster from the column, and this undoubtedly was followed by the buckling of the column.

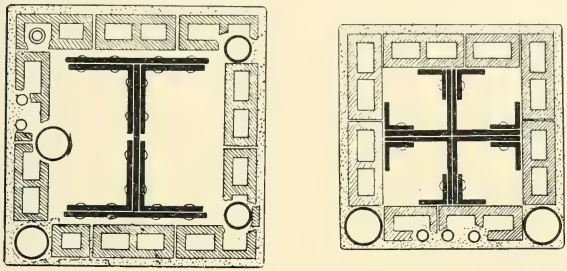

FIG. 21.—SECTION OF COLUMN FIREPROOFED WITH TERRA COTTA ENCLOSING PIPES.

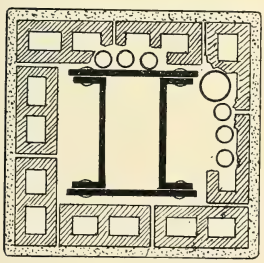

FIG. 22.—STEEL COLUMN FIREPROOFED WITH TERRA COTTA, WHOSE INNER WEBS ARE IN SOME CASES ENTIRELY CUT AWAY.

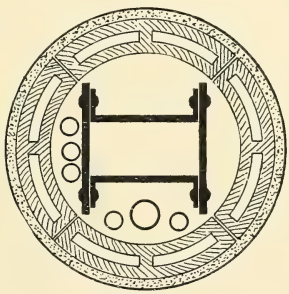

FIG. 23.—CIRCULAR TERRA COTTA FIREPROOFING OF STEEL COLUMN. EXPANSION OF PIPES WILL DESTROY FIREPROOFING.

One of the common materials for fireproofing columns is terra cotta tile. This material is in itself non-burnable, and for this reason there is a prevalent fallacious idea that the material is a

fireproofing material of the first class. The tile, as commercially manufactured, has usually webs 5/8 in. thick. Many of these tiles are cracked in the process of manufacture, and thereby weakened. When the tiles are laid up as a fire protection to a column, girder, beam or floor, the expansion of the exposed web or face of the tile being much more rapid than that of the inner webs, failure results from the cracking and falling off of the web, be-

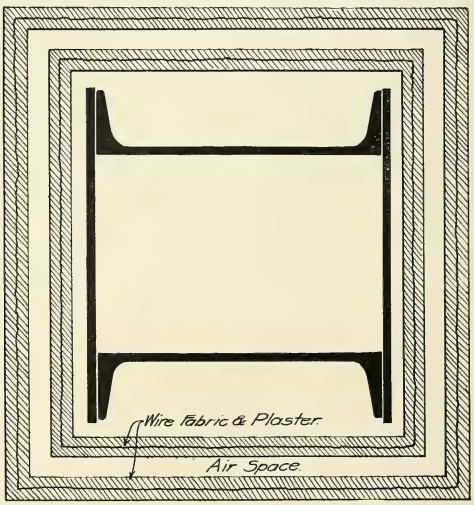

FIG. 24.—STEEL COLUMN FIREPROOFED WITH DOUBLE PLASTER ENCLOSURE LEAVING AIR SPACE.

cause of its insufficient strength to resist this expansion, which leaves the unprotected member to the mercy of the flames. Fig. 18 is a view of column in a building "fireproofed" with terra cotta tile, which has failed by buckling, as a result of the failure of this "fireproofing" from expansion under heat.

It too frequently happens that the architect, in order to secure a presentable appearance for his columns, places metal pipes and conduits of various kinds around the structural member. The

expansion of these pipes from heat causes a stripping of the fire protection of the column, with the resulting failure, as shown in Fig. 19; and this fact is still further illustrated in Fig. 20, where the buckling of the columns, following the failure of the terra cotta tile "fireproofing," caused settling of 10 or 12 ins. in the floor which they supported. A criminal method of construction is illustrated in Fig. 21, Continental Trust Building, Baltimore, where it may be seen that in order to place the "fireproofing" around the pipe, the webs of the tile were broken, thereby practically destroying the "fireproofing" value of the tile. This fact is

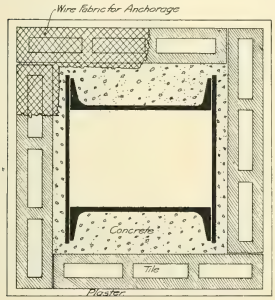

FIG. 25.—STEEL COLUMN FIREPROOFED WITH TILE AND AIR SPACE FILLED WITH CONCRETE.

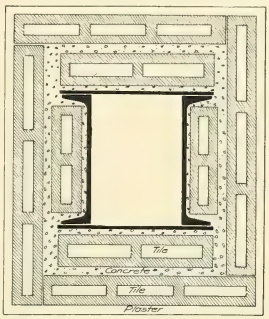

FIG. 26.—METHOD OF FIREPROOFING STEEL COLUMN WITH TILE AND CONCRETE.

still further illustrated in Fig. 22, showing that while the surface of the tile in the outer web is undamaged, the inner web is in some cases entirely destroyed. This fireproofing has then little or no value. In the section of the column shown in Fig. 23 the "fireproofing" is circular in shape, and while the tile are unimpaired, the presence of the pipes within this "fireproofing" renders the fire protection of little value, since the expansion of the pipes would very quickly strip the "fireproofing" from the column.

While a single encasement with plastered metal fabric forms a very inadequate protection, a double encasement of plastered metal fabric with an intermediate air space, as shown in the sec-

tion of the column, Fig. 24, affords a very fair fire resistance. Where this plaster is Portland cement mortar, the resistance, of course, is very much greater than where hard plaster is used, because the fire quickly dries off the water of crystallization and leaves the latter an inert mass which is easily destroyed.

A few years ago a series of tests were made in Chicago, which consisted of cutting a long concrete column into short sections. These columns were 12 ins. square, and an effort was made to show the value of the "fireproofing" by contrasting the resistance of the unprotected concrete column 12 ins. square with the behavior of a section of the same column protected with 3 ins.

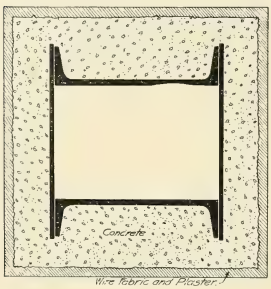

FIG. 27.—STEEL COLUMN FIREPROOFED ENTIRELY WITH CONCRETE.

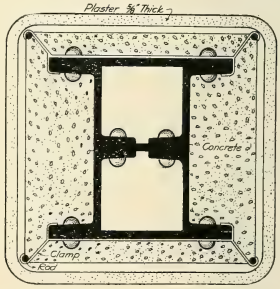

FIG. 28.—STEEL COLUMN FIREPROOFED WITH CINDER CONCRETE AND PLASTER COATING.

of terra cotta tile. As a result of these tests it was claimed the column with the terra cotta "fireproofing" had a greater resistance than the unprotected column and, therefore, terra cotta was an excellent "fireproofing" material. The fallacy of the theory evolved is whereas the tests showed that by fireproofing the column it would (as might be supposed) have a greater resistance to fire, it is also a fact that the 12-in. unprotected column would have had a greater resistance than the terra cotta "fireproofed" column if it had an additional thickness of 2 or 3 ins. of concrete.

A method of fire protection is shown in Fig. 25, where the tile encases the column and the interior space between the tile and the column is filled with concrete. Here will be seen another

method of using a wire fabric as a means of anchoring the tile to the concrete. While the tile in this particular method of fireproofing may fail under heat, the concrete will offer resistance to the fire, but the question naturally arises whether or not the column protected to the full size with concrete would not be much more efficiently fireproofed than in the method shown. Fig. 26 shows a fairly efficient method of fireproofing with tile where concrete is used to fill in the spaces. Evidently in this particular

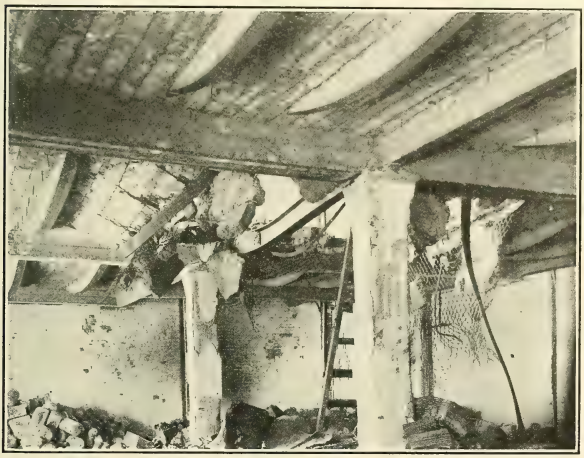

FIG. 29.—FAILURE OF FIREPROOFED FLOOR DUE TO EXPOSURE OF LOWER FLANGE OF BEAMS.

method attempt is made to utilize the hollow spaces, and it is possible this tile protected with a coat of concrete or tile would remain unchanged, but it is evident that an even more satisfactorily protected column would be obtained by the use of a very much smaller protective coating of concrete.

But, after all, experience shows that one of the best protections that can be afforded to columns is through the use of concrete, especially if there is an enclosing fabric, as shown in

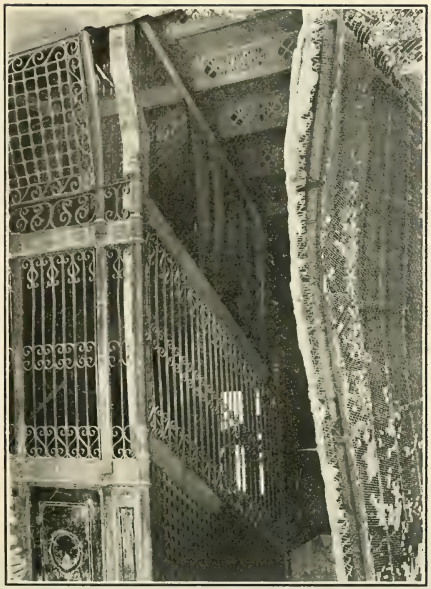

FIG. 30.—FAILURE OF PARTITION OF PLASTERED METAL FABRIC SUPPORTED BY ANGLE IRONS.

FIG. 31.—FAILURE OF TERRA COTTA PARTITION.

Fig. 27, which serves to bind together the outer plaster with the concrete itself.

In Fig. 28 the steel column is covered with cinder concrete. The ⅝-in. plaster coating on the outside of the casing of the steel will doubtless offer some protection, but even here the coating should be at least 1 in., to afford a reasonable protection.

Attention is called to Fig. 29, to point out how an otherwise good material may prove ineffective, because of defective design. The unprotected flat bands which reinforced the ribs on the under side of this floor failed in a fire and caused the collapse of several panels.

In the matter of partitions, little study is apparently given to their efficiency as fire barriers. It frequently happens that a light ½ or 1-in. channel angle iron supports two sheets of metal fabric, and the plastering on this makes a total thickness of perhaps 1 to 1½ ins. Generally hard plaster is used, and the result is that a very moderate fire soon destroys the life of the plaster, and leads to expansion of the metal fabric and the angle irons, the partition failing, as shown in Fig. 30.

Where a Portland cement plaster is used, instead of hard plaster, the damage done is not nearly as great, and if the coating of plaster be sufficiently thick it will offer a reasonable barrier to ordinary fires. A very common partition consists of 2 or 3 terra cotta tile, with a coat of plaster on either side. The heat of the fire soon destroys the plaster and causes the expansion of the tile, with the failure of some of the webs, and generally results in the collapse of the partition. A view of such a condition is shown in Fig. 31, where the failure of the partition and column protection has caused a buckling of the columns and a consequent sinking of several inches of the supported floor.

It often happens that the protecting tile fireproofing is coated with an inch or more of cement plaster, which serves as a reasonable protection to the tile. If, however, the heat is sufficiently great and prolonged this plaster is stripped off by the failure of the tile.

Unfortunately, the view given in Fig. 32 shows a condition of usual occurrence, and shows how fallacious the idea that certain forms of fireproofing may be effective. In this case the

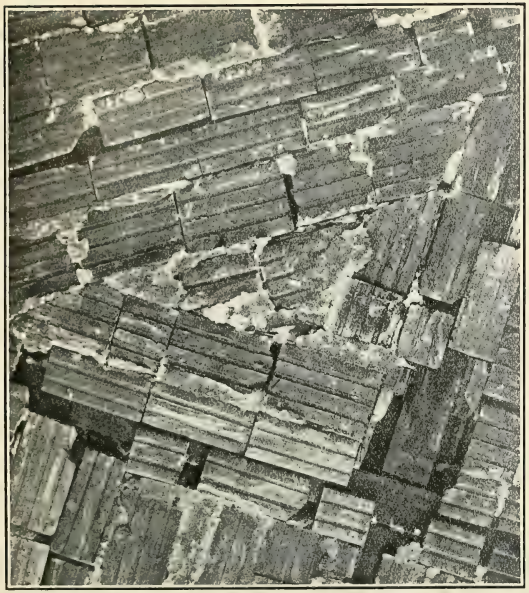

FIG. 32.—POOR WORKMANSHIP IN APPLYING TILE AS FIREPROOFING TO FLOOR, SHOWING BROKEN TILE AND UNPROTECTED STEEL MEMBERS.

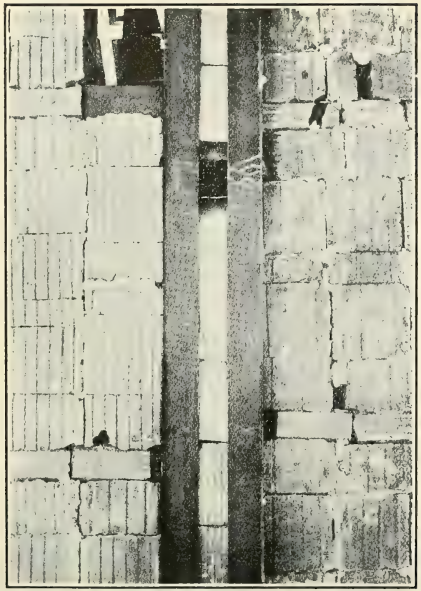

FIG. 33.—FLOOR FIREPROOFED WITH TERRA COTTA WITH UNPROTECTED FLANGES.

workman, in applying the fireproofing, has either cracked or destroyed a portion of the web, thereby destroying the insulation of the air spaces and, therefore, subjecting the inner web directly to the action of the fire.

In Fig. 33 may be seen a similar condition, but the greatest danger here is the unprotected flanges of the girders. Such views as this can be secured in almost any large city in the country, and bring out strikingly the danger of bad workmanship in destroying whatever merit there may be in this form of fire protection. The plaster too often imperfectly fills the joints of the abutting tiles, which still further offers another means by which the flames can dangerously heat up the structural member which the tile has been designed to protect.

There were a number of buildings in San Francisco in which

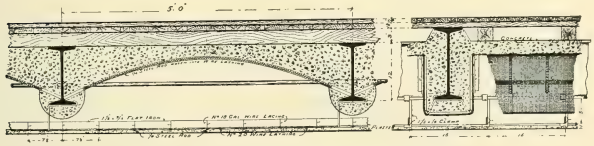

FIG. 34.—TYPE OF SUSPENDED CEILING.

the floor was protected by a suspended ceiling, and while the ceiling in some cases was destroyed because of its flimsy character, nevertheless it afforded sufficient barrier to protect the floor through the hottest part of the fire. A suspended ceiling, when properly designed, acts as an admirable fire barrier, and in case of its being destroyed or seriously damaged in a fire, can be replaced at low cost and without injury to the floor.

In Fig. 34 may be seen a type of suspended ceiling. Here the beams supporting the floor are protected by a coating of concrete, the light flimsy character of the unprotected support of the ceiling indicates one of the great weaknesses of this type of construction. The metal fabric which it supports is generally covered with a hard plaster, and the destruction of this plaster through fire allows the heat to attack the carrying member with the result that it either breaks or expands sufficiently to cause the failure of the ceiling. Where this is protected by a coating of Portland

cement mortar, it is much more efficient. It is apparent, I think, from the photograph that such a ceiling offers an admirable barrier and protects the floor from the direct action of the flames. But even under this condition it is necessary that the floor in itself be properly fireproofed, so that the suspended ceiling will form an additional barrier. It frequently happens that in an attempt to

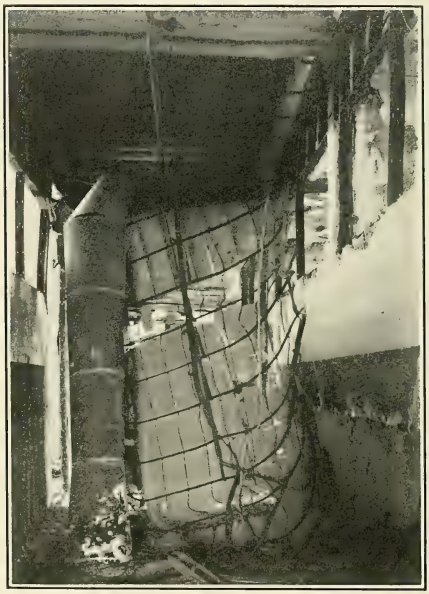

FIG. 35.—FAILURE OF SUSPENDED CEILING.

hold in place the metal fabric for the purpose of aiding in the protection of the structural member the metal comes too near the surface, or is so insufficiently protected by a coating of plaster that it serves as a means of transmitting heat directly to the member intended to be fireproofed, which action not only destroys the fireproofing, but in some cases has been sufficiently strong to soften the structural member.

In Fig. 35 may be seen a view of a suspended ceiling, which has collapsed by reason of the failure of the supporting clips, although in this particular case there was no damage to the floor itself.

Figs. 36 and 37 show two types of floor construction where

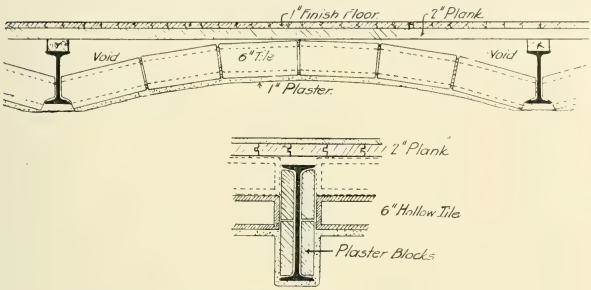

FIG. 36.—COMMON TYPE OF FLOOR CONSTRUCTION GIVING INSUFFICIENT PROTECTION.

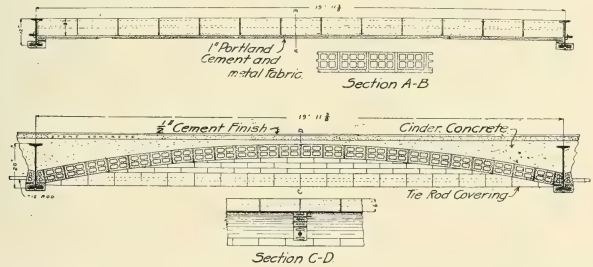

FIG. 37.—FLAT ARCH CONSTRUCTION WITH INSUFFICIENT PROTECTION OF SUPPORTING BEAMS.

there is insufficient protection. If the tile is defective and improperly laid (and has been broken so as to make defective joints and openings, as shown in Fig. 21), there is afforded a very poor protection against fire.

In Fig. 37 may be seen a section of the flat arch which is

commonly used in many of our large buildings. The behavior of fireproofing of this kind is clearly shown in Fig. 38, where the web has failed over a large area. The failure of this web was unquestionably due to the unequal expansion of the lower and

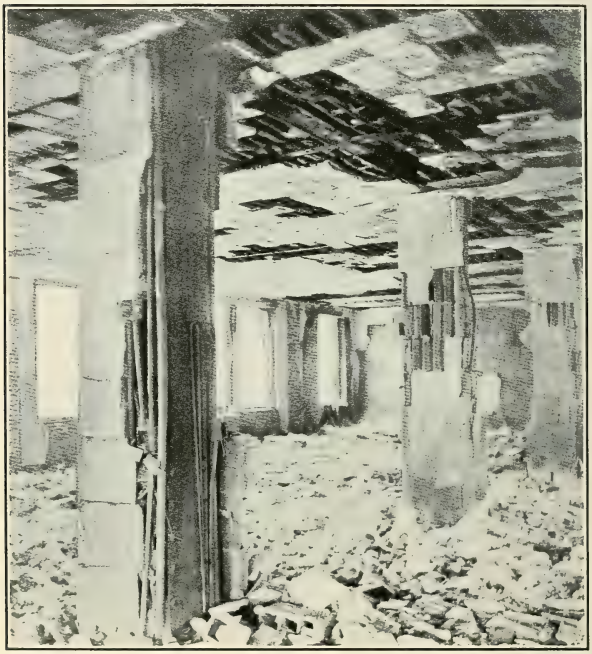

FIG. 38.—FAILURE OF TERRA COTTA FIREPROOFING THROUGH CRACKING OFF OF LOWER WEBS.

upper webs, and the connecting web being of insufficient strength, the lower web, or that subjected to the direct heat, cracked off.

Where concrete is used for the floor of a building, it is necessary that the reinforcement shall be properly protected against fire. The Joint Committee on Concrete and Reinforced Con-

crete has very properly recommended that there shall be at least 1 in. of protection to the metal in slabs, and 1½ ins. in beams and girders, and 2 ins. in columns.

The roof is perhaps a part of the structure that receives little consideration from the architect or engineer who designs it, and it too frequently happens that the roof trusses are wholly unprotected against fire, presumably on the theory that since the roof of the building it not occupied, there is little danger from

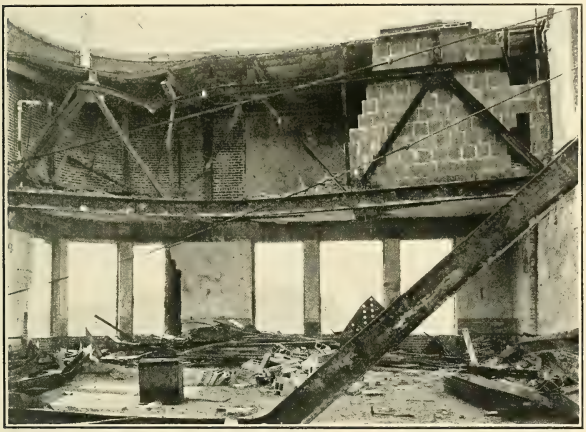

FIG. 39.—FAILURE OF UNPROTECTED STEEL ROOF TRUSS, MUTUAL LIFE BUILDING, SAN FRANCISCO.

fire, and, therefore, there is no necessity for fireproofing. It should be borne in mind, however, that the heat of the fire on the upper floor may be sufficiently great to destroy the ceiling, and the collapse of the unprotected roof trusses opens the building and leaves it to the mercy of the flames.

Fig. 39 is an excellent illustration of the failure of an unprotected steel roof truss in the Mutual Life Building, San Francisco. It is certainly as necessary, and perhaps more necessary, that the structural members of the roof shall have the same degree

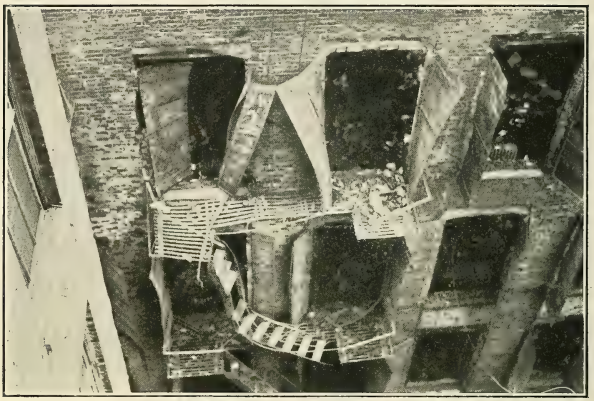

FIG. 40.—FIRE ESCAPES, ASCH BUILDING, NEW YORK, N. Y.

FIG. 41.—FIRE ESCAPES, SCHOOL BUILDING, COLLINGSWOOD, OHIO.

of fire protection as is accorded other parts of the structure. As previously pointed out, it is essential that a building shall have a maximum fire resistance both from within and without, and it is as necessary to protect the roof members, and thereby prevent their destruction by a fire in an adjoining building which would permit the fire entering, as it is to fireproof the ceiling under the roof, and thereby prevent the fire in the building destroying the roof.

Contraptions that the speaker has repeatedly criticized and scored on the ground of their lack of value, are the flimsy iron fire escapes, required by law in some of our large cities. He has frequently pointed out that it is difficult under ordinary conditions, unaffected by the hysteria that a fire produces, to walk down these steps safely, and how much more difficult it is to utilize them in case of fire, especially if they be unprotected, and pass windows where flames are breaking forth. It frequently happens that the fire destroys the fire escapes themselves, as shown in Fig. 40, the fire escapes of the Asch Building, which proved of little value as a means of egress for the occupants of this ill-fated building. Fig. 41 shows the Collingswood, Ohio, School Building, with its iron fire escapes.

It is high time that the fallacy regarding these flimsy structures should be exploded, and the laws which legalize them be repealed, and new ones enacted which will absolutely prohibit the use of such structures.

It is necessary that there shall be a proper means of exit from a building in the event of fire. In Europe the number of such exits is controlled by the number of occupants of the building. It frequently happens that enclosed stairways are protected by plain glass windows, and that the stairways consist of cast iron horses and marble treads; their valuelessness is illustrated clearly in Fig. 42, which shows the practically total wreck of such an enclosed stairway.

The Asch Building fire drew attention to a condition which is too prevalent in New York City, and while all the discussion was directed against the Asch Building, as a bad type of structure, we should not lose sight of the fact that within a stone's throw of this building are numerous other structures infinitely worse than the Asch Building as regards their fire-resistive properties.

It so happened that a fire occurred in the Asch Building, which brought this in the center of the stage, but those who inspected the conditions in the vicinity of the Asch Building realize that there are many buildings of infinitely poorer construction than this building—as defective as proved its provisions against a fire.

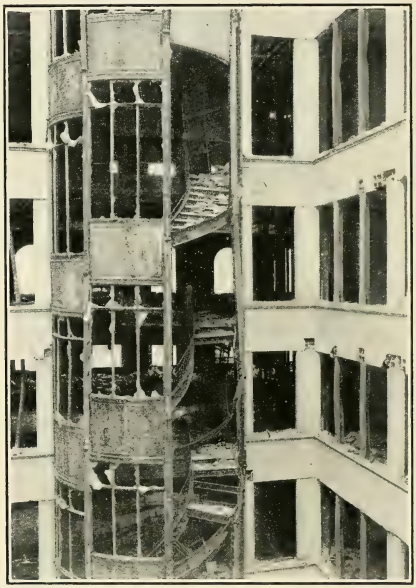

FIG. 42.—FAILURE OF POORLY PROTECTED STAIRWAY.

Probably the best type of fire escape is the fire tower. There are a number of types of such buildings, and two forms are illustrated in Figs. 43 and 44. The principle involved is that the fire stairway proper shall be shut off from the floors of the building, and that the entrance to this fire tower shall be from the exterior. In the case of the type shown in Fig. 43, it is necessary to go from the building to an open air vestibule, and then through a

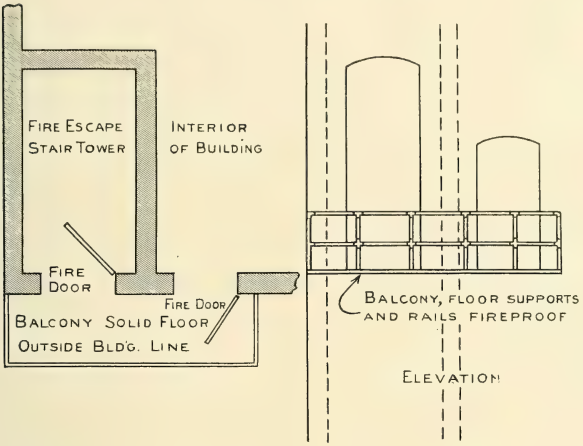

FIG. 43.—MODEL FIRE TOWER WITH EXTERIOR BALCONY.

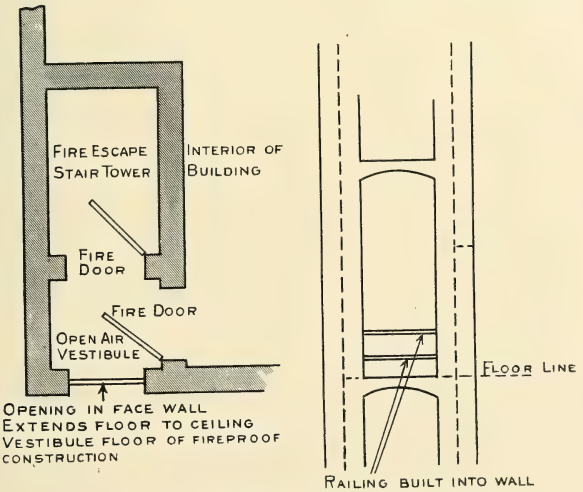

FIG. 44.—MODEL FIRE TOWER WITH INTERIOR VESTIBULE.

door into the fire tower. In this way the rush of flames into the fire tower is prevented, and enables the occupants of the building to escape with safety.

The foregoing illustrates a few of the many fallacies which prevail in the matter of fireproofing and fire protection, because of a lack of information on the subject. It is to be hoped that the fire-resistive properties of the various building materials and of various systems of protection shall be developed so as to make the subject so well known as to make possible a general knowledge of what constitutes first-class fire protection. While tests of the individual materials afford information as to their relative fire-resistive values, and while the work done in tests of separate panels affords perhaps a little better knowledge of their properties, nevertheless the real test is that of an actual building where the full force of expansion, due to the heat generated by the burning contents, is felt; such expansion produces the destruction which has been indicated in many of the foregoing photographs.

Fire protection is a study of the expansion of materials under heat, and in order to properly analyze the subject, the fundamental requirement is a knowledge of the expansion of the various materials and their rates of heat conductivity. Fire protection consists in applying to the structural skeleton a coating of suitable material of low heat conductivity and of sufficient thickness to afford it the requisite protection from the heat that is likely to be generated by the combustion of the contents of the building. It is, therefore, evident that greater protection must be afforded the contents of a warehouse than would be necessary in the case of an office building, and it is important that our building laws shall classify the various requirements as regards fireproofing in accordance with the character of the occupancy.

It should be a matter of gratification to the members of this Association, that cement is constantly gaining favor as a splendid material for fireproofing purposes, and it should be the duty of every member to study the question and become so familiar with the fire-resistive properties of this material as to see that it is properly applied for such purposes; it behooves the Association to establish the value of this material for fireproofing purposes so that structures may be intelligently fireproofed. The failure of

a structure improperly fireproofed injures the standing of such a material, even though it be evident that it was a matter of bad design or workmanship, or both.

The inadequate laws of this country are gradually undergoing a change, and are being replaced by more effective ones. In this connection, we should not criticize our city fathers too harshly, for we should bear in mind that the laws of European countries, which we cite as models, are the result of the gradual evolution of laws originally as inadequate as many of the laws of this country. It is, after all, a matter of education and publicity, and the more the subject is discussed, the more the principles of fire protection are pointed out and the better they are understood, the sooner will be created that degree of knowledge which will lead to an intelligent revision of our building laws.

THE NATIONAL FIRE PROTECTION ASSOCIATION AND ITS WORK.

By W. H. Merrill.*

The National Fire Protection Association, as its name indicates and as its Articles of Association state, was organized to promote the science and improve the methods of fire protection and prevention; to obtain and circulate information on these subjects, and to secure the co-operation of its members in establishing proper safeguards against loss of life and property by fire.

From a small beginning made about 15 years ago, the organization now comprises a large membership, and is unique, I believe, in listing ninety of the principal National Institutes, Societies and Associations and Insurance Boards in America as active members. Among these we are proud to point to the National Association of Cement Users, for, as a fellow townsman of mine stated at your last annual convention, the cement user is a true conservator, in that he draws upon a material which would otherwise remain inert and useless for all time.

The National Fire Protection Association stands for conservation of the created resources of America. We believe that the fire waste is of great importance as affecting the material welfare of the nation as a whole, and is of direct obligatory and pecuniary interest to each one of us having families to protect and property to preserve for our family's use. Each loss by fire is a drain upon the material resources of the country, irrecoverable through the collection of insurance.

A fine business building with its merchandise is an asset in the created resources of the nation. The building burns, the contents are a total loss. The asset is wiped out. Taxing the fortunate for the benefit of the unfortunate will not bring it back. This is elemental. Consequently, the value of our annual ash heap is an annual debit against the wealth of the people of the United States of America. How much do we lose annually from this source? We know about conspicuous cases, such as the loss

*President, National Fire Protection Association, Chicago, Ill.

of Seventy-five Million Dollars at Baltimore, and of Three Hundred and Fifty Million Dollars at San Francisco,—but what is the annual average taken over a period of years? The average annual loss by fire in the United States during the past 10 years has been over Two Hundred Million Dollars, and during the past 5 years has averaged over Two Hundred and Fifty Million Dollars.

An average loss for the last 5 years of Two Hundred and Fifty Millions means a loss of about Thirty Thousand Dollars for each hour. One thousand Two Hundred and Sixty Million of dollars in the products of human toil have been wiped out of existence in the brief space of 5 years. The values represented by the fire waste would more than pay the stock dividends of all the railroads. It exceeds the revenue of the National Postoffice; it is greater than the production of gold or silver.

The annual number of fires in American cities averages forty for each 10,000 of population, as compared to eight for each 10,000 of population in European cities. The annual per capita loss in Australia, Denmark, France, Germany, Italy and Switzerland varies from 12 cents in Italy to 49 cents in Germany, with an average of 33 cents; in the United States the average is $2.47. Berlin has a population of Three Million. Its average annual fire loss is One Hundred and Seventy Thousand Dollars. Chicago, with its population of Two Million, burns up annually nearly Five Million Dollars of values. We destroy more property by fire than do any four nations in Europe.

Is there anything beyond this Two Hundred and Fifty Million Dollars which we should include as America's contribution to Waste by Fire? I am not among those who add the cost of all fire departments and other things which are not waste in any sense to their computations on this subject. I am of the humanitarians, who feel that America's greatest contribution consists of her men, her women and her children burned alive as sacrifices to the ignorance, carelessness and greed of her citizens.

Before the dollar marks I would write the list of those who perished at the Iroquois Theatre in Chicago, on the steamer General Slocum at New York, in the hall at Boyertown, at the school in Collingswood, in the mine at Cherry, and in lesser numbers at many other places. The remembrance of their cries and the

horrors which they suffered should appeal to us more strongly than the money loss, be it ever so large. What sacrifices would not be made could the children burned at the Iroquois, at Boyertown, at Collingswood, and the men of Cherry be returned to their families. They would include some greater things than money, I am sure.

Taking all of these factors into consideration, is this not then a subject of real present and vital importance to the American people?

Many of us have discussed quite at length and formed fixed opinions as to the need of the country in building up a navy to protect us against the sometime invasion of a foreign foe. Are we consistent in neglecting to consider the necessity for protection against an ever present foe within our borders, which is annually killing more people than all our wars of recent years and destroying property values equivalent to the price of a fairly large new navy every year?

We have joined in thoughtful consideration of many plans which have been devised covering various proposed relations between the people and the railroads. Have we thoughtfully considered any measures for the relief of the people from an annual debit item against their resources in excess of the amount all of the railroads have ever returned to their stockholders in their most prosperous years?

Government expenditures in excess of One Billion Dollars per year have been adversely criticised as extravagant, but little discussion has been attracted by the total loss of a quarter of this amount annually through waste by fire.

Waste and extravagance are important factors in the increased cost of living, which is now a matter of general complaint. The epitome of recklessness in expenditure is supposed to be represented by the person who uses bills of large denominations with which to light his cigars. This individual employs this means to advertise the fact that he has money to burn. Our National fire waste seems to classify the people as a whole with this individual. Is it pleasant to conceive of him as the typical American? In the economic scale it is unimportant whether or not the burning of half a dozen grain elevators affects the price of bread, or the destruction of a number of cold storage plants affects the prices

of other food products. It is the destruction which counts, whether it be wheat, beef, dry goods or other values. For that which the individual destroys by fire under insurance taxation the nation must pay.

The National Fire Protection Association has been steadily engaged in an effort to reduce in some measure this enormous and disproportionate waste. Through our active member, the National Board of Fire Underwriters, a model building code has been prepared and freely distributed to officers of all municipalities who could be interested. Through the same organization expert fire protection engineers visit all of the principal cities of the country, and make extensive reports on the conflagration conditions. In our own organization we have appointed a large number of Committees, each one of which has perfected model specifications in various branches of fire protection engineering and in the field covered by fire prevention. These have been issued in large editions, and have been adopted as standard requirements by a number of municipalities and by many insurance organizations. In Chicago laboratories have been erected where examinations and tests are made and reports issued on devices, materials, system and appliances having a bearing on the fire hazard. This institution, the Underwriters' Laboratories, has erected and occupies what is generally believed to be the best model of fireproof building construction in America, and up to the present time over 5000 reports on various subjects and appliances coming before it have been promulgated. The findings of these reports are generally accepted by the authorities throughout the United States and Canada.

Special Committees of the National Fire Protection Association are working on the subject of fireproof construction, including Concrete and Reinforced Concrete, and on a good many other subjects which would interest each individual member of the National Association of Cement Users.

Of the specifications coming under the jurisdiction of this Association, possibly the one best known is the National Electrical Code, which covers the safeguarding of electrical equipment, and which has been adopted as the law of more municipalities and the standard of a greater number of other organizations with authority than any other single set of engineering specifications in existence to-day.

Another publication covers specifications for automatic sprinkler installations, outlining in detail the methods to be followed in equipping buildings with this most useful and successful extinguishing agent. Publications of a similar character, which have been distributed in editions of many thousands, and which are freely used by property owners, architects, insurance boards, and others interested, include specifications for the construction and installation of fire doors and fire windows, fire shutters, electrical signaling systems affecting the fire hazard, valves and hydrants, hose, suggestions for organizing private departments, and various forms of building construction. Specifications for safeguarding hazardous appliances and systems coming under the general head of explosives and combustibles have been issued, including films for moving picture machines, systems for storing fluids which at ordinary temperatures give off inflammable vapors, acetylene gas machines, gasolene vapor gas lighting machines, lamps and systems, engines and stoves, rules for waste cans, ash cans, refuse barrels, fire pails and safety cans for benzine and gasolene, storage and use of fuel oil, kerosene oil pressure systems, and also suggestion for protection against lightning.

We are issuing from time to time circulars and copy for newspapers on both the common and the periodical causes of fire, such as the hazards of Christmas decorations and displays, the menace of the match, the hazards of defective chimneys and flues, and also the proper season warnings as to celebrations of the National holidays, particularly the Fourth of July.

All of this points among other things, of course, to the necessity for doing away with combustible material in building construction and equipment, and adds its force to the arguments you have to advance for the use of Cement. I feel that many of your members would be interested to receive regularly the publications of the National Fire Protection Association, and to enjoy the other privileges of membership in that organization, just as I feel it is decidedly to my advantage to enjoy membership in the National Association of Cement Users.

Quoting again from my fellow townsman, Mr. Northrop*:

One of the greatest demands made upon science in these modern days is the effort to conserve our resources, and in that connection you

*For complete Address see *Proceedings*, Vol. VI, p. 31.—ED.

gentlemen may be characterized as genuine conservators. I am told that you make everything which man needs during and after his life with cement; that from the time we are ushered in upon this earthly scene we are likely to be cradled in cement, to live in houses of cement to do life's work through instrumentalities of cement, and finally, when "we have shuffled off this mortal coil," our remains are to be laid away in coffins of cement, to await the sound of the final trumpet. You are, therefore, no less benefactors in the race than he who makes two blades of grass grow where but one had grown before. When you commit our mortal dust to the dust of mother earth in caskets of dust, you leave the trees, which else had been wrought into our last covering, to wave in the forests, or to be used for living and not the dead. You are all the more true conservators in that you draw upon a material which otherwise would remain inert and useless for all time. Now, we behold this otherwise useless material rising into splendid edifices, into beautiful cities, and taking on many forms expressive of our civilization.

From this it follows that in the work of the National Fire Protection Association we may count upon your continued support and helpfulness, and we on our part may co-operate as we have through membership on such of your committees as the services of our individual representatives may be deemed useful, thus bringing together through thoroughly neighborly relations our common agencies for the general good,—to the end that by such united resistance some headway may be made against what many of us believe to be the greatest enemy of mankind in America to-day,—the evil and uncontrolled spirit of the flame.

SOME THERMAL PROPERTIES OF CONCRETE.

By Charles L. Norton.*

During the last three years a series of experiments have been carried on in the Laboratory of Heat Measurements of the Massachusetts Institute of Technology, having for their object the study of those physical properties of Portland cement concrete which affect its value as a fire resistant material. While these researches are by no means complete, it is perhaps of interest at this time to discuss some of the results obtained.

It was proposed at the outset to make a study of the various physical properties of concrete over as wide a range of temperatures as possible, and among those selected were the following:

1. Coefficient of linear expansion.
2. Diminution of mechanical strength after heating.
3. Specific heat.
4. Coefficient of thermal conductivity.

1. Coefficient of Linear Expansion.

The measurements of the coefficient of linear expansion are practically complete, but the uncertainty attaching to some of the high temperature values, though slight, makes it seem wise not to report in detail until one or two check measurements can be made. This uncertainty arises from the great difficulty which attends the measurement of the temperature of the interior of a large block of cement concrete so as to be sure of its absolute uniformity at high temperatures. Thermal junctions embedded in the block at distances of 3 ins. from one another are not sufficiently close to give the desired precision.

The method adopted for the measurements of the elongation caused by heating was the common so-called telescope method. The specimens in the shape of 6-in. or 10-in. cubes were slowly

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heated in a double gas muffle or an electric resistance furnace. The temperature of the furnace and of a number of points in the concrete was taken by means of platinum-rhodium couples.

Near the furnace were mounted two telescopes, Figs. 1 and 2, which might be sighted through holes in the furnace wall upon reference points on the surface of the block. At low temperatures an arc light and a system of mirrors were used to furnish adequate illumination. One of the telescopes was provided with a micrometer

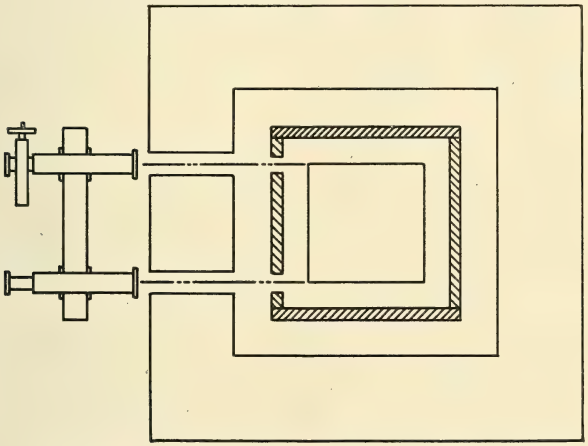

FIG. 1.—DIAGRAM OF APPARATUS FOR MEASURING COEFFICIENT OF EXPANSION OF CONCRETE.

eye piece by means of which a movement of the reference mark of .0001 in. could be measured.

The values obtained at low temperatures agree very well with the commonly accepted value of .0000055 for the elongation per unit of length per degree Fahrenheit. Apparently this value increases slightly up to 575 deg. F. Above this point the coefficient becomes smaller; at 1,500 deg. F. the coefficient becomes zero, and above this point slightly negative. On cooling the blocks which had been heated to 1,500 deg., they did not return

to their original dimension, their permanent elongation being about 75 per cent. of their maximum elongation. On a second heating the specimens all returned to the same dimensions, there being no sensible permanent elongation resulting from the second heating.

All of the specimens tested for expansion were of stone con-

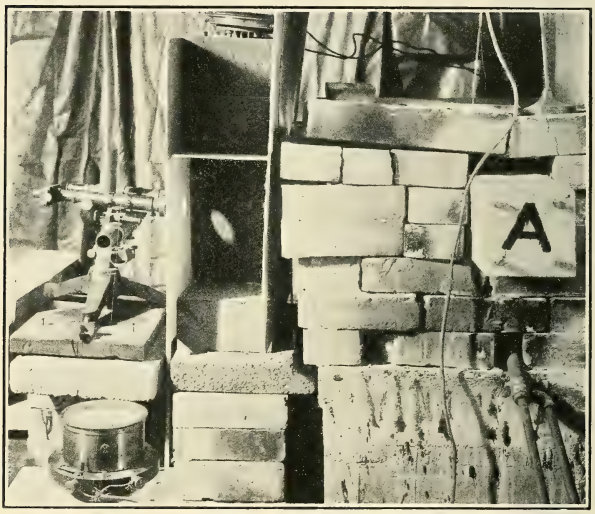

FIG. 2.—APPARATUS FOR THE MEASUREMENT OF THE COEFFICIENT OF EXPANSION OF CONCRETE, SET UP FOR TEST.

crete, of the proportions 1:2:5. The stone was clean, the sand sharp, the cement of good quality, and every precaution was taken to secure a concrete of the first order. The specimens weighed on the average 150 lbs. per cu. ft.

2. DIMINUTION OF MECHANICAL STRENGTH AFTER HEATING.

In order to study the effect of high temperatures upon the compressive strength of concrete several scores of 6-in. cubes were made and allowed to set for 90 days or slightly longer.

These blocks were then heated for different lengths of time at different temperatures in a gas furnace similar to that used for the expansion experiments.

The cubes which were not heated showed an average compressive strength of 2,700 lbs. per sq. in. Those which were heated to 932 deg. for two hours lost only 15 per cent. of their original strength, and when this temperature was prolonged for four hours the loss was 43 per cent. If the blocks were heated to 1,500 deg. for two hours the loss of compressive strength was 38 per cent., and at the end of four hours the loss was 66 per cent. The most important information secured from these tests was to the effect that even after exposure to a fire in which the concrete block, small in size, was subjected to flames at a temperature of 1,500 deg. for four hours, it was still able to carry twice its normal

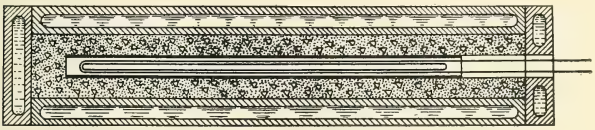

FIG. 3.—CROSS-SECTION OF PLATE TESTING APPARATUS.

safe load in compression. The larger beams with their great masses are not often exposed for so long a time to so great a hazard.

It should be noted, however, that there was a considerably greater deformation under load of the heated blocks than of those not heated.

A large number of small beams were next made, some with and some without reinforcement; most of these were either $6 \times 6 \times 48$ ins. or $8 \times 8 \times 48$ ins. The specimens which were reinforced contained four $\frac{1}{2}$-in. round steel rods situated near the corners, equi-distant from the two faces of the beam. In some the distance from the reinforcement to the face of the beam was 1 in. and in others $1\frac{1}{2}$ in. A few beams had a 2-in. protection to the reinforcement. Not many of these specimens have been tested as yet, but, from a few, we are able to get a general idea of what happens to beams of this type in case of exposure to fire.

Three beams for instance, each $6 \times 6 \times 48$ ins., in which the reinforcing rods were 1 in. from the face of the beams, were broken by center load, the first beam not having been heated at all, the second heated for one hour in a fire that fused the surface of the concrete, and the third being similarly heated for two hours. The beam which was not heated broke under a load of 5,700 lbs.; the second, heated for one hour, broke at 2,750 lbs., while the third, heated for two hours, broke at 1,950 lbs. I consider this a most remarkable showing under severe conditions.

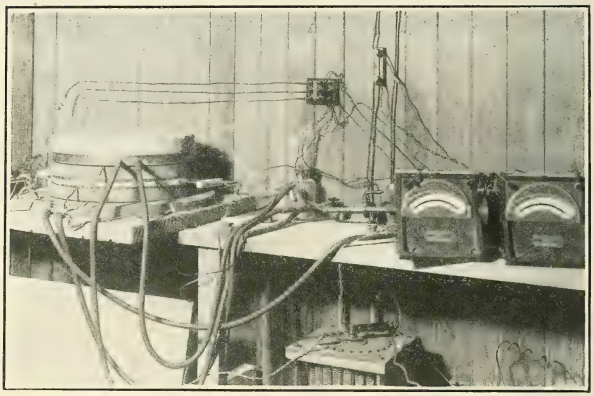

FIG. 4.—VIEW OF PLATE TESTING APPARATUS FOR DETERMINING THERMAL CONDUCTIVITY OF CONCRETE.

It should be borne in mind that these small beams were so slow in cooling down that they were in effect heated much longer than the time mentioned. The flames, moreover, surrounded the beams on all sides.

Several similar tests were made with concrete beams of the same dimensions, $6 \times 6 \times 48$ ins., with no reinforcement. These were heated for two and four hours at 1,700 deg. F. After this exposure all of the beams were very frail, not one of them showed any cracking or spalling, however. It was not considered necessary to break any of these beams, as it was apparent that the

concrete on the tension side of the beam was much weakened. Some of the more recent experiments made on beams $8 \times 8 \times 48$ ins. show that these larger beams are much less weakened, proportionally, as would be expected to be the case.

A series of similar beams were next made up of cinder concrete, the proportions of the mixture being 1 : 2 : 5. A portion of these were mixed with clean cinders, which showed upon analysis but little carbon; a second part was mixed with cinders to which

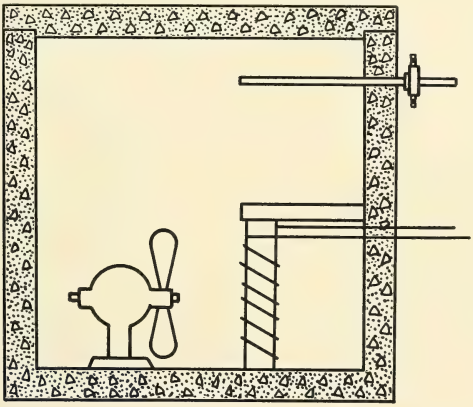

FIG. 5.—ELECTRICALLY HEATED CUBE FOR TESTING CONDUCTIVITY.

10 per cent. of fine bituminous coal had been added and the other beams were mixed with cinder, to which had been added 25 per cent. of fine coal. The 25 per cent. mixture can be disposed of in a word. When once thoroughly heated it burned until it fell to pieces. With the 10 per cent. mixture, however, no such action occurred; there was no indication that the concrete would support its own combustion even for a short time. It was apparent that the 10 per cent. mixture was not so good a fire-resistive material as that which contained no added carbon. From the few specimens containing less than 10 per cent. which have been

examined up to the present, it seems probable that the safe limit is close to 5 per cent. More information is now being secured on this point.

3. SPECIFIC HEAT.

The study of the specific heat of concrete was made by the ordinary calorimeter method, the "method of mixtures" of Regnault. Specimens of the concrete, usually fragments of the larger test pieces, were heated slowly in an electric resistance furnace to the desired temperature and then plunged into the calorimeter. The weight of the water and its rise in temperature give the amount of heat given off by the body in cooling.

Extraordinary precautions were taken in getting the exact average temperature of the specimen in the furnace, and to insure its rapid transfer to the calorimeter. In most of the experiments a double calorimeter was used so that the specimen did not come in contact with the water of the calorimeter, so that any evolution of heat by hydration of the cement was avoided. Table I gives the specific heat of concrete.

TABLE I.—SPECIFIC HEAT.

Temperature Degrees Fahrenheit	Stone Concrete 1–2–5	Stone Concrete 1–2–4	Cinder Concrete 1-2-4
72 to 212	.156	.154	.—
72 to 372	.192	.190	.180
72 to 1172	.201	.210	.206
72 to 1472	.219	.214	.218
72 to 2172	.—	.—	.—

4. COEFFICIENT OF THERMAL CONDUCTIVITY.

The measurements of thermal conductivity were made by a number of methods and have taken far more time and energy than all the others put together. The thermal conductivity is that property which determines how rapidly heat will travel through a substance and how rapidly therefore objects beyond will be heated by transmission. The conductivity becomes of prime importance in all questions of protection of the metal in reinforced

concrete buildings. There is very little data to be found as to this important property of any of the common materials of engineering and such data as are to be found are not at all concordant. As to the conductivity of concrete or its variation with temperature and with composition, practically nothing has been known.

The methods adopted for the measurements will be here described in outline only. The formula showing the relation of the temperature upon the two sides of a plate to the amount of heat which would flow through it is as follows:

$$Q = \frac{K\ (t_1 - t_2)\ s\ A}{d}$$

$$\text{or } K = \frac{Q\ d}{(t_1 - t_2)\ A\ s}$$

where K = the coefficient of thermal conductivity dependent upon the nature of the material and its temperature.

Q = the quantity of heat flowing through the plate in the area measured.

A = the area.

t_1 = the temperature of the hotter side of the plate.

t_2 = the temperature of the cooler side of the plate.

d = the thickness of the plate.

s = time during which Q units flow through the area A.

The formula will be seen to be merely an expression of the following relations, that the flow of heat is proportional to the area, to the temperature, and the time, and that it is inversely proportional to the thickness.

After spending many months in developing other methods the electrical method used by the writer for the past 15 years in studying the flow of heat through steam pipe coverings was adopted. The value Q of the heat flowing was determined by supplying the heat by means of the heating of a conductor carrying a current of electricity. By measuring the electrical energy supplied the quantity of heat developed may be known with great precision. Further, if this heat is passed through the plate under test and into a calorimeter on the far side, a check upon the value of Q may be had. For the determination of the temperature

difference thermal couples, resistance thermometers, and mercury thermometers were used, but thermal junctions made of thin strips of copper and nickel or of platinum and platinum-rhodium were generally found most serviceable.

The apparatus used for the lower temperatures consisted of a thin, electrically heated plate, to the two sides and edges of which concrete could be applied. Figs. 3 and 4. Outside of the concrete there were then placed heavy copper or brass plates which could be kept at a constant temperature by an internal circulation of water. Thermal junctions were placed at several points on each surface of each concrete plate. The electrical input was measured

FIG. 6.—CROSS-SECTION OF HIGH TEMPERATURE TESTING APPARATUS.

by calibrated Weston instruments, and calibrated thermal junctions gave the value of the temperature difference to the nearest one-one hundredth of a degree. For the thickness, numerous measurements were made with a pair of flat-nosed calipers and averaged.

It was necessary to keep this apparatus running for several days before it could be balanced, that is, before the rate of flow of heat outward through the plates became constant and equal to the electrical input. The number of measurements has now reached many hundreds, and it seems apparent that the precision of the method depends on the accuracy with which the thickness

of the specimens can be maintained constant. Otherwise the accuracy is better than 1 per cent. This apparatus has been named by my assistants the "flat plate tester" and perhaps it is well to so designate it here.

Later, in order to make tests on plates as thick as some of the walls in common use, another method was adopted. Cubical boxes 36 ins. in outside dimension were built with walls of several thicknesses. Inside the boxes were placed electric heaters which served to raise the inside surface to a temperature above that of the surroundings. A sketch of the apparatus is shown in Fig. 5. A small fan served to keep the air in the box stirred to insure uniformity of temperature throughout. The boxes were tightly

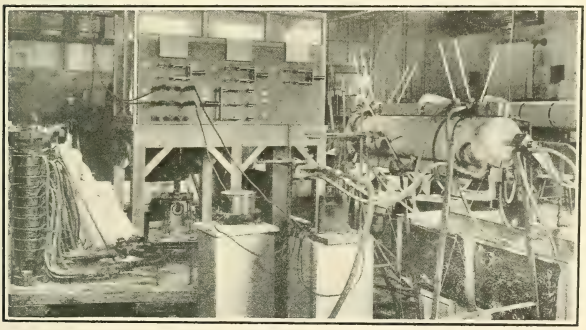

FIG. 7.—GENERAL VIEW OF HIGH TEMPERATURE TESTING APPARATUS.

sealed. The power supplied to both heater and fan was measured as before. Mercury thermometers and thermal junctions, as well as a Callender recording resistance thermometer, were used to measure the difference in the temperatures inside and outside of the box. Measurements have been made on scores of specimens and are practically identical with those obtained by the flat plate tester.

For the high temperatures a modification of the entire process was found necessary. The concrete to be tested was cast in the form of a cylinder on the outer surface of and concentric with a steel bar which could be heated to a high temperature by the passage of a heavy current. Outside of the cylinder of concrete

was applied a closely fitting "continuous" calorimeter. A cross section of the apparatus is given in Fig. 6. The temperatures of the bar and of the calorimeter were measured by thermal junctions, and the amount of water and its rise in temperature gave the value of Q. A general view of the apparatus during test is shown in Fig. 7.

In order to guard against the uncertainty of the temperature at the ends of the bar, the calorimeter was made so as to enclose only about one-half the length of the bar, the rest being covered

TABLE II.—COEFFICIENT OF THERMAL CONDUCTIVITY OF CONCRETE.

Temperature of hot side of plate. Degrees		Mixture	Coefficient. Calories per 1 deg. C. per sq. cm. per cm. per second.	Coefficient. B. T. U. per 1 deg. F. per sq. ft. per in. thick per 24 hrs.
Centigrade	Fahrenheit			
35	95	Stone 1-2-5	.00216	150.
50	122	Stone 1-2-4 not tamped	.00110 to .00160	76. to 114.
50	95	Cinder 1-2-4	.00081	56.
200	392	Stone 1-2-4	.0021	146.
400	752	Stone 1-2-4	.0022	153.
500	932	Stone 1-2-4	.0023	160.
1000	1832	Stone 1-2-4	.0027	188.
1100	2012	Stone 1-2-4	.0029	202.

by guard rings similar to the calorimeter but without provision for the measurement of the quantity of water.

The heating of the bars required a considerable amount of special apparatus, since it was necessary to provide a current of upwards of 2,000 amperes for the high temperatures, and to be able to vary its amount to any desired value below that point. For this purpose there were installed three 15 k.w. transformers connected on the primary side with a three-phase 2,300 volt circuit. By means of divided secondaries and a rather elaborate arrangement of switches, the secondary voltage could be varied from 190 volts down to 55 volts. This secondary voltage was applied to the primary of a second step-down transformer, whose secondary was divided into twenty coils. By means of a switchboard the entire output of the transformer could be had at almost any desired low voltage. This enables us to heat bars insulated by materials of different composition and of different thicknesses

to any desired temperature up to 2,800 deg. F. With this arrangement both the steel and the concrete can easily be melted.

The results obtained are given in Table II. It is to be regretted that there is no uniformity of practice as to the units to be adopted in reporting the measure of effectiveness of insulators. While the physicist renders his report in calories per square centimeter, per centimeter thickness, per 1 deg. Centigrade per second, the steam engineer confines his observations to B. T. U. per hour, per square foot, per inch of thickness, per 1 deg. F. and the refrigerating engineer reports on the basis of a 24 hour time unit. I have even seen a report in terms of hogsheads of water raised to the boiling point, time not stated.

A brief comparison of these values with those for other materials may be interesting.

The specific heat of concrete is slightly less than that of either red brick or fire brick, hence the same amount of heat needed to raise the temperature of a pound of brick is about 10 per cent. more than for a pound of concrete. But the density of concrete is enough greater than that of brick to raise the heat capacity of a cubic foot of concrete above that of brick. The difference is not large, however.

While the expansion data is not complete, it seems clear that for a time after the beginning of exposure to fire, the concrete and its reinforcement will expand at much the same rate, but that the further expansion of the surface will not proceed at so rapid a rate. This will tend to reduce the stresses which the expansion of the heated surface would otherwise set up in the cooler interior. It is perhaps because of the failure of the concrete to return to its original dimensions that the small amount of surface cracking found after a fire is due.

The experiments made with coal and cinder mixtures indicate the necessity of added care in the selection of cinders for this purpose.

The table of thermal conductivities gives us data as to the rate at which heat will travel through concrete. It is interesting to note the great difference between the tamped and the untamped concretes made from stone. The one was as porous as possible,

and the other as dense. One transmits nearly twice as much heat as the other. The cinder concrete, as is commonly believed, is much better as a heat insulator than the stone concrete, being nearly three times as effective as the denser stone concrete in retarding the flow of heat. It may be interesting to call attention to the heat insulation afforded by other materials. The best of the commercial articles used for this purpose is compressed cork, which is nearly 25 times as effective as stone concrete. Steel on the other hand transmits heat from 75 to 100 times as fast as the densest of the stone concrete.

DISCUSSION.

Mr. Ira H. Woolson.—I would like to ask what Mr. Norton has found to be the average percentage of coal in the average kind of cinders obtained in Boston, and as to the percentage of coal or carbon contained therein. Were hard or soft coal cinders used and did they come from boilers? Mr. Woolson.

Mr. Charles L. Norton.—The highest value I have recollection of was 7 per cent., and many were very much below that, but my knowledge is limited, not having the data of the tests at hand. Boiler cinders were used almost altogether. Mr. Norton.

Mr. Cloyd M. Chapman.—What does Mr. Norton think would be the effect if the carbon had been in lumps rather than in a finely powdered state? As a rule, the cinder I have seen and examined with a view of powdering, contains carbon of the size of the head of a pin, even larger pieces of coke, and represents quite a different condition from finely distributed and uniformly powdered carbon. Mr. Chapman.

Mr. Norton.—It was endeavored to study this point, but the size of the specimens was limited, beams 8×8 ins. and 10×10 ins. being the largest that could properly be heated in the laboratory, and pieces, several of them as large as $1\frac{1}{2}$ ins., were used. What happened was that the large pieces, about $1\frac{1}{2}$ ins. in diameter, near the middle of a beam 8×8 ins., would almost invariably cause a blister and a splitting off of a conchoidal plate in front of the piece of coal. But I believe the examination of a beam only five or six times as large as one lump of coal really ought not to be counted as more than a suggestion. That is, in a reinforced concrete beam 18×24 ins., the effect of a piece of coal with a maximum dimension of $1\frac{1}{2}$ or 2 ins. might have been very different from that in the small beams used. Mr. Norton.

Mr. L. C. Wason.—At what temperature does cement disintegrate and what temperature will it stand for an indefinite period, and will it disintegrate very rapidly at a white heat? Mr. Wason.

Mr. Norton. MR. NORTON.—I should not like to be quoted on this particular question, although my business is heat measurements and I suppose my laboratory is equipped with more delicate apparatus for that purpose than any other laboratory in this country. I am not certain what the temperature is inside of these concrete specimens when the temperature upon the outside is 1,500, 1,600 or 1,800 degs., unless a thermal junction is placed in that particular spot. That is, the velocity of heat as it travels through the concrete is so low that while I am perfectly sure as to what happens to the surface of the specimen when it is exposed to fire at a certain temperature, I am not sure that I know anything more than that only the very skin surface of the specimen is exposed to that temperature, and it has recently been necessary to revise a most elaborate set of figures on this point.

In a general way, at a temperature of 500 degs., which is not red hot, there is a tendency for decomposition to begin in such portion of the cement as is at that temperature. It is not a serious matter until the neighborhood of 900 degs., red heat, is reached, and above that disintegration proceeds with considerable rapidity. Blocks which have been exposed to 1,500 degs. for 2 hrs., show a loss in compressive strength of 38 per cent.; and if they are exposed 4 hrs. they show a loss of 66 per cent.

In speaking of 1,500 degs. it is meant that the furnace was 1,500 degs., but the center of the block probably did not reach 1,400 degs. Those that were heated to 1,000 degs. for 2 hrs. showed, even red hot, a loss of about 15 per cent. of their original compressive strength; but if that temperature was prolonged to 4 hrs. a loss of about 45 per cent. was shown.

Mr. Woolson. MR. WOOLSON.—How soon after removal from the furnace were the blocks subjected to the compressive strength tests?

Mr. Norton. MR. NORTON.—Some of the blocks were tested inside of 15 mins. and some of them after 2 or 3 hrs. They were wrapped up and kept hot during the transfer, which was made as quickly as possible, but none of them were broken while they were red hot upon the outside.

Mr. Woolson. MR. WOOLSON.—In an investigation made a few years ago on this subject, I found that the blocks heated to 1,500 degs. or thereabouts for 3 or 4 hrs., if left to stand for 3 or 4 days before

testing, had practically lost all their strength. At 1,500 to 1,700 degs. for 4 hrs., and then standing several days, a decided disintegration took place in the blocks. Mr. Woolson.

Mr. Norton.—We did lay some of the blocks aside for 3 months and have them now at the end of a year or so. I remember distinctly Mr. Woolson's figures. We found those kept in a damp place showed some such tendency as the blocks that he used. And further, we found that some stones showed this tendency much more than others. We did not notice the effect you mention, although we were, on account of your notes on that subject, rather looking out for it. Perhaps you may have caused a chemical decomposition of the cement which we did not cause. The blocks which were treated in the electric furnace, where they were not heated in contact with anything but air, have not shown as much tendency to air-slack as those heated in the gas furnace. But after spending hundreds of dollars we have just begun to get on the right track. It has proved rather a longer research than either Mr. Woolson or myself had any idea of. Mr. Norton.

Mr. Sanford E. Thompson.—Has the interior of blocks which had been exposed to heat ever been tested after removing the soft, disintegrated concrete from the surface, so as to determine the effect of the heat upon the strength of the concrete in the interior which is apparently unaffected? My reason for asking this question is because it is obvious that the disintegration of the surface concrete, even for a fraction of an inch, will reduce the effective area of the specimen so that one would expect a lower strength due simply to this cause alone. For example, suppose a 4-in. cube is disintegrated on all sides to a depth of ½ in. This will reduce the effective area of cross section from 16 sq. ins. to 9 sq. ins.; in other words, the disintegration of a 4-in. cube to a depth of only ½ in. will reduce the crushing strength of the total specimen nearly one-half, even if the interior is absolutely good. Mr. Thompson.

Mr. Norton.—Each one of these cubical blocks was cast with a 3/16-in. hole running from one end to the center so that an electric thermometer could be placed in the bottom of the hole. After breaking them in the compression machine it was perfectly evident that about a 4-in. cube within the middle of the 6-in. cube had not been affected by fire. So that even there by Mr. Norton.

Mr. Norton. pounding, crushing or breaking no disintegration could be noticed. On the other hand, the nature of the compression failure, that is the familiar hour glass break shown by these cubes, indicated that the outside had fallen off just as suggested, that there was a sort of coating on the outside which split off under very moderate loads, or cracked so it would fall off and the center of the block would take the whole load. It was evident that the top and bottom faces were softer and that the compression load coming down upon it did not bear squarely upon the hard nucleus of the block. So the figures undoubtedly would have been higher than suggested, but it was not possible to obtain enough cubes out of the middle of any of them for crushing tests to get the figures desired.

Mr. Thompson. Mr. Thompson.—In accordance with this, in a large specimen or in practice where masses are apt to be larger than in laboratory work, the percentage of loss of strength would be much lower than in small specimens. In other words, the loss of strength may be largely due to the reduction in the effective area by the disintegration of the surface, while the interior may perhaps remain substantially intact.

The President. The President.—The Chair would like to call attention to the tests made by the United States Geological Survey at the Underwriters' Laboratory in Chicago, which consisted of testing some 8×13-in. concrete beams of cinder concrete containing up to 25 per cent. of unburned coal in the shape of rather large lumps. These beams were exposed on one face for a period of 2 hrs. to a temperature of 1,800 degs. The results are interesting and bear out in a general way the behavior of cinder concrete.

Where the particles of coal are exposed to the direct action of the flame the coal burned even though a small point only was exposed. This exposed point emitted enough oxygen to consume the carbon. Where, however, there was a coating of cinder over the particles of coal it was apparently undamaged by the heat treatment. In other words, only the exposed particles of coal were consumed. If I recall these tests correctly the temperature mark did not extend to more than 1¼ ins. from the exposed surface. The beams were placed with their largest dimension

vertical and the back of the beam did not become sensibly hot during the tests although only 8 ins. from the exposed surface. The President.

I think the question of the resistance of the materials is a question of the relative heat conductivity and where there is a large mass that will absorb the heat the damage done to the structure is not nearly as great as where you have a small mass in which the heat can attack it from four sides. In a building the under surface of the floor is not subjected to the same intensity of action as in the case of a beam or girder where the heat can attack it from three sides.

In studying the effect of the heat conductivity while these small test pieces do form a relative measure of the fire resistive properties of the material, nevertheless, the real study of the value of such materials must be made on large masses and under conditions in which they are actually used.

Mr. Woolson.—I would like to add a little testimony in regard to the behavior of cinder concrete under fire tests, which bears out the statement of Mr. Norton. Mr. Woolson.

While at Columbia University I constructed a building for the fire testing of floors, which was about 18×22 ft. square. This building was made entirely of cinder concrete, the ordinary run of boiler cinders with a 1 : 3 : 5 mixture. On top of the structure were six chimneys 1 ft. sq. or a little larger inside, and the walls of the chimneys were 4 ins. thick. That structure has been subjected to 24 hrs. of firing with an average temperature of 1,700 degs. for the whole period, and at least half of that time the temperature was around 2,000 degs. To-day the building is standing in as perfect condition as it was the day it was built, barring some few temperature cracks which, of course, are due to that very excessively severe treatment. The insides of the 4-in. chimneys are just as perfect to-day as they were when the forms were taken out, and the imprint of every board used in the forms can be seen where the concrete has not been hit by streams of water played upon the floors under test.

I speak of this because I think it is a very remarkable case. The cinders were hard coal cinders, and from the appearance of the outside of the building, which of course was the same as the inside, I should think it would be safe to say that there was nearly

Mr. Woolson. 10 per cent. of crushed coal in the mixture. I made no test of the cinders.

Mr. Miller. MR. RUDOLPH P. MILLER.—I am a great believer in cinder concrete for fireproofing purposes, but it is necessary to use satisfactory materials, and a large proportion of unburned coal is detrimental to the cinders as a fireproofing material.

In supporting this I want to refer to one of the fire tests, made under the supervision of the Bureau of Buildings of New York City, which proved a failure. I felt so certain of the result of the test to be made that I did not go myself to see it but sent my assistant with instructions to let me know how the test progressed so that I might be there at the finish. I do not believe it was more than an hour when I received word that the floor under test had collapsed and the test was a failure. The walls of the test structure had also been built of cinder concrete, and when I got there, much to my surprise, the walls were afire due to the large amount of coal in them.

Mr. Norton. MR. NORTON.—The reason cinder concrete does all that Mr. Woolson states lies along this line. After measuring the heat flowing through 1 in. of cinder concrete, it was found to be only about 1/3 as much as through stone concrete. That is, after there has been a 1,700-deg. fire going up the chimney for 4 or 5 hrs. the temperature 1 in. away is not raised 1/3 as far above the room temperature as it would, had that been made of ordinary stone concrete, granite, etc. The efficiency lies right there. Cinder concrete refuses, although its surface is white hot, to transmit the heat to the interior.

Mr. Cummings. MR. ROBERT A. CUMMINGS.—Some 5 or 6 years ago I made a test in which a brick of Portland cement and granulated blast furnace slag mixed in the proportion of 1:3 and a brick of Portland cement and ordinary washed river sand mixed in the same proportions, were placed side by side in the furnace of a steam boiler. In about 15 minutes the brick of river sand and cement had crumpled to pieces and fluxed, and in 25 minutes the slag cinder brick was intact but of a white heat. It was then removed from the furnace and placed in a bucket of cold water. The result of the immersion was the disintegration of the edges of the brick only, the remaining part being apparently uninjured.

Mr. Norton. Mr. Norton.—Only one or two experiments were made with granulated slag cinder concrete. Its conductivity is very high. I am not prepared to say it is better than ordinary cinder concrete or worse, but it is distinctly a better insulator than stone concrete. It is hardly necessary to state this if there is considered the multitude of tiny air spaces which run all through the slag. Partly on account of chemical disturbance in the reinforcing metal started by some of the slag used, and partly because of the difficulty of obtaining homogeneous lots in Boston, we made very few tests. It is a good non-conductor and better than stone concrete.

Mr. Chapman. Mr. Chapman.—What percentage of unburned coal or coke in a cinder concrete would be considered permissible or safe for an exposure that was not strictly for the purpose of fireproofing? Take the case of a manufacturing establishment with little fire hazard, the cinder to be used in the roof only, cinder being chosen because it requires less structural frame for support. Waterproofing to be of tar and gravel or slate or tile, but not designed as a high grade of fireproofing, simply a light, cheap, fire resisting slab. Again, would not a good quality of crushed stone concrete be expected to endure without injury a temperature of luminous gases, of perhaps 800 or 900 or possibly 1,000 degs.?

The President. The President.—The Chair does not care to answer that question. He is in sympathy with the recommendations of the Committee that cinder concrete should not be permitted for purposes other than fireproofing and that for such purposes it should be free from ash or unburned coal. Doubtless there are some cases where economical conditions might permit the use of cinders containing unburned coal when it is used as an aggregate in fairly large masses and the particles of coal are kept away from the exposed surface by a protective coating of mortar. It would probably, under these conditions, withstand light heat without serious disruption, but if the heat is rather intense and prolonged it would be a very undesirable material for the purpose.

Mr. Woolson. Mr. Woolson.—Unfortunately I have no data as to definite amounts of coal in the cinder concrete used. It seemed to me that 7 per cent. would be perfectly safe, but that is merely an opinion and is not based upon actual figures. By examination of different samples made in a crude way, that is, spreading out and

Mr. Woolson. estimating the amount of coal without actual measurement or calculation, I did not find over approximately 7 per cent. I know that those cinders were perfectly safe and had no hesitation in using them. I never have happened to see any boiler cinders in this city that in my opinion contained coal enough to injure the concrete so far as fire resistance is concerned.

I might say in regard to the particular building Mr. Miller referred to as a failure, that the same did fall in 20 minutes after the fire started, but there was a very large amount of unburned coal in the cinder concrete. The building was constructed in such a poor manner that upon lighting the fire smoke immediately came out through all the walls, as if they had been sieves. It is no doubt understood that there was some structural weakness in the building. There was coal enough to burn and the walls actually did continue to burn after we turned on a hose and put the fire out in the interior of the building.

The President. The President.—The presence of coal in cinder concrete is certainly a detriment under conditions where it is likely to be subjected to high, prolonged temperatures. It is possible to use cinders containing coal in a structure which is not subjected to high temperatures providing the coal is kept away from the exposed surface.

Mr. Woolson. Mr. Woolson.—The lower the percentage of coal the better the concrete. There is one thing which must be taken into account and that is the large percentage of what appears to be coal in cinder concrete is not coal at all, but black slate. The interior of the building referred to on looking at the walls at the present time, there appear to be large patches of coal the size of a finger nail here and there all over the building, which are really nothing but black slate which the fire has not changed in color.

The President. The President.—Cinder that makes a desirable fireproofing material is in the nature of a clinker. In burning coal in a furnace there is, of course, a lack of perfect combustion resulting in a percentage of unburned coal. The Chair feels that this material is not desirable for fireproofing purposes. He admits, however, that it can be used and is used for such purposes, but this is not of course evidence that it is a desirable material for fireproofing purposes. The coal ought not be there and is an undesirable

aggregate and cinders which contain a percentage of coal should not be used for fireproofing purposes. It is a fact that coal cannot burn without oxygen to support combustion. If concrete containing cinders with a percentage of coal is used for fireproofing purposes and effort is made to keep the coal away from the surface, it affords a moderate amount of protection providing the heat is neither intense nor prolonged. If the heat is high and prolonged the coal would eventually be consumed and in the process of oxidation produce a swelling which will be apt to disrupt the concrete. **The President.**

MR. G. H. BRAZIER.—It seems to me rather an important thing that if 7 per cent. of coal is safe, then at least half of that, 3½ or possibly 5 per cent., is allowable. Specifications frequently state that cinder concrete shall be free from unburned coal. It has been my observation that no cinder is absolutely free, any more than the sand is absolutely free from some impurity. It does seem to me that the specifications should state the allowable percentage, just as with sand a certain amount of organic matter is specified as permissible. **Mr. Brazier.**

THE PRESIDENT.—Will the speaker cite the case where the specifications for sand permit organic matter being present? **The President.**

MR. BRAZIER.—I just saw a specification of a contract covering, I should judge, a $60,000 concrete dam, on which bids were received and where the specifications definitely stated that the sand should carry not to exceed 3 per cent. of organic matter. The specifications were drawn by a Boston engineer whose name I do not remember, but the fact remains. **Mr. Brazier.**

THE PRESIDENT.—The Chair desires to say that there is no standard specification which permits the presence of organic matter. I agree with the speaker that it is not possible to secure sand which is absolutely free from organic matter, but it is certainly a fact that the specifications ought not recognize the presence of organic material, leaving it a matter for the discretion of the engineer to decide whether a small percentage of organic matter is permissible. There is a tendency always to mistake organic matter for the finely dissolved clay. The former is detrimental, while the latter, if not permitted to excess, tends to improve the quality of the concrete. **The President.**

Mr. Cummings. MR. CUMMINGS.—I question the value of reinforced cinder concrete at all. I do not see any justification for its use, not even as a fireproofing material, when it is in contact with unprotected metal work. Is it not a fact that cinder in any form is detrimental to the reinforcement? And if that is true, which I believe it to be, it is an objectionable material and should be prohibited in all reinforced concrete work. I am aware that in cities, cinder ashes can be obtained for nothing or for the cost of hauling, so that the cost of construction is reduced. But, in my judgment the quality of this material is an added reason for its prohibition. Further, is it not a fact that the gross weight of a cinder concrete structure is as great as that in which the usual quality of aggregates are used? Because the unit stresses in the concrete are smaller and consequently the bulk is proportionately increased.

Cinder concrete is desirable as a filling material on account of its light weight. But if it is to be thus brought in contact with reinforcement or structural steel work, the metal must be covered by an impermeable coating. This may be secured by dipping the reinforcement several times in a bath of neat cement.

Mr. Chapman. MR. CHAPMAN.—We have made some experiments of the effect of cinder concrete upon imbedded clean iron and have found that unless the cinder concrete was dampened again after it had set and dried out, the corrosive action of the cinders on the iron was less than the protective action of the cement, and that there was no deterioration as long as it was kept dry. But let it get wet and rusting began immediately and progressed. A dry cinder concrete had no effect on steel for the length of time that it was tested, which was a year or so. The test was rather carefully made, that is, the plates were thoroughly cleaned of mill scale and rust before imbedding and were carefully examined after they were taken out of the concrete. Rusting had not started within a year, so probably it would not have gone very far in a period of years. The indications were that iron is safe from rust in cinder concrete if the concrete is kept dry. In the case of the same cinder kept damp by standing in a shallow pan of water, there was a rusting of the iron within a few months.

The President. THE PRESIDENT.—In San Francisco there was probably as large a percentage of cinder concrete used for fireproofing as will

be found in any city in this country. There are numerous instances where the imbedded metal fabric used for reinforcing the floor slabs had been completely disintegrated; due, I think, always entirely to the presence of sulphides in the coal which formed the medium which produced the corrosion that entirely destroyed the metal fabric. The President.

Mr. Miller.—I think that the corrosive action is largely due to the coal from which the cinder comes. As has been stated, in San Francisco they have had that experience. Mr. Cummings refers to the experience in Pittsburgh. We have used a great deal of cinder concrete in New York for fireproofing and for floor construction and I have yet to hear of the first instance of corrosion in steel due to the fact that cinders were used for the concrete. I think it is in part due to the fact that cement enters into the concrete mixture, and has a protective effect on the steel. We have had the experience of the serious rusting of gas pipes where they have been laid in dry cinders, but I have yet to know of the first case in New York of the corrosion of steel in cinder concrete. Mr. Miller.

As to its being a satisfactory fireproofing material, the experience here must lead to the conclusion that it is the best kind of fireproofing material.

Mr. Woolson has referred to a structure in which the walls were of cinder concrete. In all the cases that have come under my observation where cinder concrete was used for the wall construction and subjected to the severe fire test required by the New York law, there has been practically no deterioration in the walls. Every brick or stone concrete wall used in such test structures has shown deterioration. This is largely due to the low heat conductivity of brick work and stone concrete. The interior of the wall, which is exposed to the fire, becomes heated and expands; the exterior remains comparatively cool, does not expand so much, causing the wall to crack. That is the case in both the brick wall and in the stone concrete wall. I have never seen it in the cinder concrete wall. The porous character of the cinder concrete, which allows of an easier adjustment of the material, would, I think, account for this fact.

REPORT OF THE COMMITTEE ON FIREPROOFING.

The Committee has continued work as outlined in its report last year and owing to the difficulty of securing adequate information must submit a Report of Progress.

The schedule of information desired on the efficiency of fireproofing materials has been sent out, and an endeavor made to obtain as much information as possible, especially in the case of actual fires. There have been so few fires in concrete buildings and the effect on the structural members has been such as to render little information to the Committee. Owing to the detail data required, both of the effect of a fire and of the design of the structure subjected to fire, there is great difficulty in securing data of value in preparing specifications for fireproofing.

The Committee will endeavor during the current year to collect all information possible, so that at the next Convention a suggested specification may be presented if at all possible.

Respectfully submitted on behalf of the Committee:

RUDOLPH P. MILLER,

Chairman.

REPORT OF THE COMMITTEE ON INSURANCE.

The Committee on Insurance has continued work along the line of investigation started two years ago, and the results are given in tables and in comments on letters received from owners. The work has been done by correspondence with owners, by conference with rating associations and by study of reports of the United States Government and the fire protection associations of the country.

One additional feature of the work this year has been the investigation of insurance rates of cement hollow block buildings, which are separated from monolithic construction.

Part I.

The result of study of the reports from owners of concrete buildings and further investigation lead the Committee to the following conclusions regarding rating of concrete risks:

No Definite Established Rates. There are no definite rates which can be considered base rates for any fireproof structures, either concrete or fireproofed steel frame.

More Interest in Concrete Buildings by Insurance Companies. The Insurance Rating Associations have shown a decided increase in interest in reinforced concrete construction during the year.

Present Rates too High. Members of the Committee are unanimous in the opinion that in the majority of cases the owners of concrete buildings reported are paying too high a rate, because of the lack of knowledge on the part of the owner of methods of securing lower rates.

The Committee advises the owners of concrete buildings to study this annual excess charge from the standpoint of elimination of waste with the same diligence that they are now studying the elimination of waste in manufacturing.

Cause of High Rates. The main cause for the high rates reported is the lack of proper installation of recognized fire prevention equipment, together with the lack of information on the

part of the owner regarding the construction details, in advance of making his investment: that is, the owners have not equipped their buildings in accordance with the best known practice conducive to low rates of insurance.

Effect of Fire Protection. Rates have been low where buildings are acceptable in both stock and mutual insurance companies.

New Risks Acceptable to the Mutual Companies. Mutual Companies have accepted risks in congested portions of cities, which would not have been acceptable had the buildings not been of reinforced concrete construction.

Important Details Affecting Rates. The following list of details of construction have been most emphasized by the Rating Associations, and should be investigated by owners before letting contracts for buildings where the cost of insurance plays an important part in the investment:—

1. Sprinkler equipment with ample water supply.
2. Water tight floors with proper drains or scuppers and vertical openings guarded by automatic closing devices.
3. Proper cut-off walls with automatic closing doors and automatic fireproof windows. (Size of divisions of the building to depend on the inflammability of the contents.)
4. Notification systems for out-break of fires.
5. Fire fighting apparatus independent of city or town fire departments.
6. Stair and elevator wells properly cut off by automatic closing doors and fireproof apparatus.
7. In addition to the above details, a typical rating slip gives some 50 items of equipment, construction or management, any or all of which affect the rate obtainable.

Re-adjustment of Rates Based on Standard Practice. The Committee feels that there is a demand for preliminary information regarding insurance rates on proposed buildings, and that if by co-operation between the members of this Association and the different Rating Associations, approximate estimates of the rates obtainable for a given building, depending on the class of con-

struction adopted, can be secured, often better buildings will result than is at present the case.

Frequently ultimate cost (varying with the insurance interest and depreciation charges) is more important than initial cost, which means poor construction.

The members of this Association should unite on a definite plea for a new classification, with an accompanying re-adjustment of insurance rates by all the Rating Associations of the United States. As a step towards attaining this end the Committee recommends that the following standards of practice should be persistently advocated.

1. Standard No. 4* of this Association, entitled *Standard Building Regulations for the Use of Reinforced Concrete;* Standard† No. 7, entitled *Recommended Practice. Report of Committee on Reinforced Concrete;* Report of the *Joint Committee on Concrete and Reinforced Concrete.*‡
2. Use of standard fireproof equipment in all concrete buildings. This will avoid the common error of assuming that the insurance rates will be low because the building is fireproof, regardless, however, of the important details covering the protection of its contents.
3. Get the approval of the Insurance Rating Associations of the plans for proposed buildings before contracts are awarded, and wherever possible secure competition between the Stock and Mutual Companies.

Tables I and II show very definitely the difference between the average rate paid on concrete buildings, as reported by owners to the Association, and the average cost to insure high class reinforced concrete risks, acceptable to Mutual Companies. It is believed that these tables will be of interest.

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* *Proceedings,* Vol. VI, p. 349.

† *Proceedings,* Vol. V, p. 454.

‡ *Transactions,* Am. Soc. C. E., Vol. LXVI, p. 431. *Proceedings,* Am. Soc. Test. Mats., Vol. IX, p. 226. *Proceedings,* Am. Ry. Eng. and M. W. Assn., Vol. XI, p. 994.

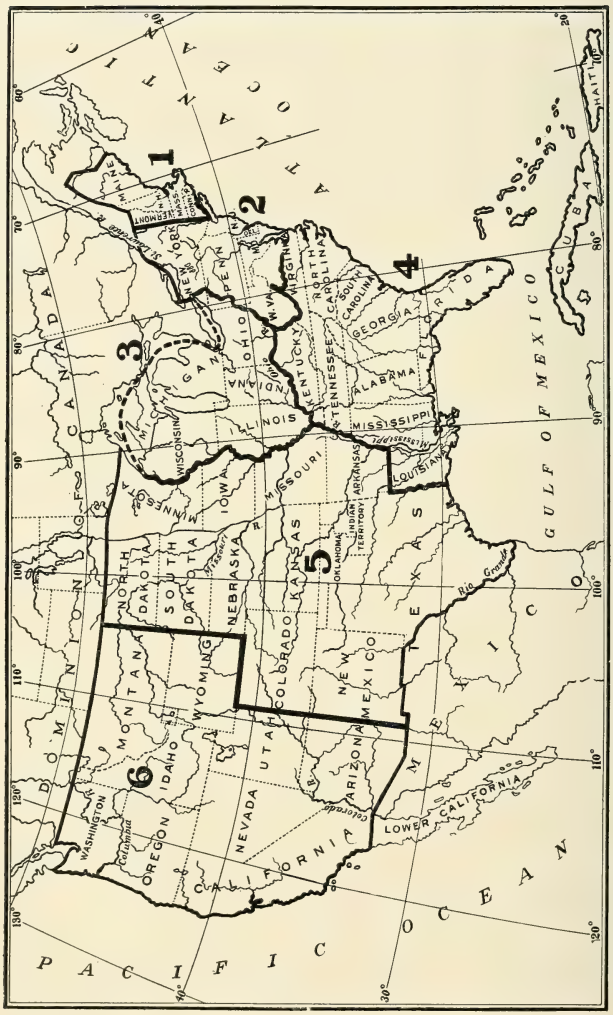

FIG. 1.—MAP SHOWING INSURANCE DISTRICTS.

The data for Table I is taken from information received from the owners of 262 concrete buildings.

Table II was prepared by the President of one of the senior Mutual Insurance Companies for use in this report, and is based

TABLE I.—SUMMARY OF RATES OF INSURANCE ON CONCRETE BUILDINGS.

Use of Building.	Minimum Rate	Maximum Rate	Average Rate Reported.	Number of Risks not Insured.	Number of Risks Included.
Light manufacturing	.04	3.00	.48	7	59
Heavy "	.016	1.50	.34	22	45
Office buildings	.06	1.09	.53	8	30
Retail stores	.12	.90	.30	—	12
Wholesale stores	.13	.70	.28	2	17
Warehouses	.09	1.75	.39	11	49

TABLE II.—AVERAGE COST OF INSURANCE ON REINFORCED CONCRETE BUILDINGS AND CONTENTS WITH SPRINKLER EQUIPMENT.

Agricultural Implement Factories	$ .06
Automobile Factories	.045
*Bakeries	.07
*Clothing Factories	.06
Lithographing and Printing Establishments	.055
Machine Shops	.04
*Metal-Working Plants, without Sprinklers	.06
Rubber Factories	.055
Shoe Factories	.05
Textile Mills	.05
Tobacco Factories	.06
Wood-working Plants	.07

* Risks not ordinarily open to competitive rates unless of superior construction.

Under this type of construction the insurance may be accepted not only at a lower rate, but under a considerable less valuation, probably resulting as a whole in reducing the cost of the insurance from 40 to 50 per cent. over the ordinary type of factory and warehouse buildings.

on the average dividends for the last five years of companies carrying risks, as noted in this report. These companies insure mostly manufacturing plants and warehouses, and represent a total insurance of $2,500,000,000.

Table III shows insurance rates for concrete block buildings, reported to the Committee.

Explanation of Table III.

Districts. In each class the rates have been distributed ac-

cording to the location of the buildings in the six districts shown on the map, Fig. 1.

TABLE III.—INSURANCE RATES ON CONCRETE BLOCK BUILDINGS.

	Districts.											
	1		2		3		4		5		6	
	B	C	B	C	B	C	B	C	B	C	B	C
Church	1.20	..	.24	.24	1.20	..	..	..	..	..	..	..
Dwellings	..	..	.48	.48	.40	.40	..	..	1.00	1.00	..	..
"	..	..	.11	1.40	.70	..	..	..	1.40	.45	..	..
"	..	..	1.00	—[3]	.70	..	..	..	..	..	..	..
"	..	..	1.00	.20	.70	..	..	..	..	..	..	..
"	..	..	.10	.10	.70	..	..	..	..	..	..	..
"	..	..	.30	—	.70	..	..	..	..	..	..	..
"	..	..	—	—	—	..	..	..	..	..	..	..
"	..	..	.085	.09	.60	..	..	..	..	..	..	..
"	..	..	.20	.20	1.30	1.30	..	..	..	..	..	..
"	..	..	.20	.20	1.20	1.20	..	..	..	..	..	..
"	..	..	1.00	1.00	.25	.25	..	..	..	..	..	..
"	..	..	—	—	.20	—	..	..	..	..	..	..
"	..	..	3.60	3.00	—	—	..	..	..	..	..	..
"	..	..	—	—	1.00	1.00	..	..	..	..	..	..
"	..	..	.35	3.72	..	..	..	..	..	..	..	..
"	..	..	.25	.15	..	..	..	..	..	..	..	..
"	..	..	.25	.15	..	..	..	..	..	..	..	..
"	..	..	.25	.15	..	..	..	..	..	..	..	..
"	..	..	.75	..	..	..	..	..	..	..	..	..
"	..	..	.15	.20	..	..	..	..	..	..	..	..
"	..	..	.75	1.00	..	..	..	..	..	..	..	..
"	..	..	.75	..	..	..	..	..	..	..	..	..
Garages	..	..	—	—	.70	..	..	..	..	..	..	..
"	..	..	.75	..	—	—	..	..	2.95	2.95	..	..
"	..	..	..	..	..	..	..	..	2.30	2.30	..	..
Manufacturing, light	..	..	3.37	..	.08	.08	..	..	2.99	2.99	..	..
" "	..	..	1.00	.75	.17	.17	..	..	..	..	..	..
" "	..	..	..	..	1.08	1.08	..	..	..	..	..	..
" heavy	..	..	—	.17	.17	..	.10	.10	..	..	..	..
" "	..	..	—	1.08	1.08	..	..	..	..	..	..	..
Office building	—	—	..	..	..	..	..	..	..	..	..	..
" "	..	..	.50	..	..	..	..	..	..	..	..	..
Public "	..	..	.70	..	..	..	..	..	..	..	..	..
Retail stores	..	..	.82	1.58	..	..	..	..	..	..	..	..
" "	..	..	1.00	1.00	1.88	1.75	..	..	..	..	..	..
" "	..	..	1.75	1.75	..	..	..	..	..	..	..	..
School	..	..	.24	—	..	..	..	..	..	..	..	..
Warehouses	..	..	1.00	.75	..	..	..	..	2.25	2.25	..	..
"	..	..	..	..	..	..	..	..	2.45	2.45	..	..

Column B covers the rate on the buildings only.

Column C covers the rate on the contents only of each building.

— indicates no insurance carried.

The original letters from which all the tables were prepared are on file at the office of the Association in Philadelphia, Pa.

PART II.

The Committee desires to show pictorially different buildings in which the construction plays an important part as regards the ability of the building to resist fire.

Figs. 2, 3 and 4 show a typical frame dwelling during framing

period; a reinforced concrete monolithic dwelling; and a concrete block dwelling.

Figs. 5, 6 and 7 show a factory building of the old wood frame type during erection; a brick building known as "mill construction" or "slow burning construction," during period of construction; and a reinforced concrete factory building in which one of the most dangerous risks has been insured at a very low rate.

Figs. 8 and 9 show a mercantile building of reinforced concrete and a mercantile building of structural steel, fire-proofed.

Part III.

Standard equipment recognized as of value in securing low rates.

The following list of publications covering rules, suggestions and specifications, together with reports of fires, will be found of value for securing low rates:

1. Pamphlets published by the National Board of Fire Underwriters, Chicago, Illinois.

Acetylene Gas Machines and Storage of Calcium Carbide	1910
Coal Gas Producers (pressure and suction systems)	1908
Electric Wiring and Apparatus, Installation Rules (Nat'l Electrical Code)	1909
Electrical Fittings, List of Approved	April, 1910
Fire Departments, Private	1902
Fire Doors and Shutters	1908
Fire Extinguishers, Chemical (for other than fire department use)	1909
Fire Hose, for fire department use	1909
Fire Hose, for private department mill yard use	1909
Fire Hose, Unlined Linen, for use inside buildings	1909
Fire Pumps (steam)	1908
Fire Pumps (electric)	1904
Fire Pumps (centrifugal)	1909
Fire Pumps (rotary)	1909
Gas and Gasolene Engines	1910

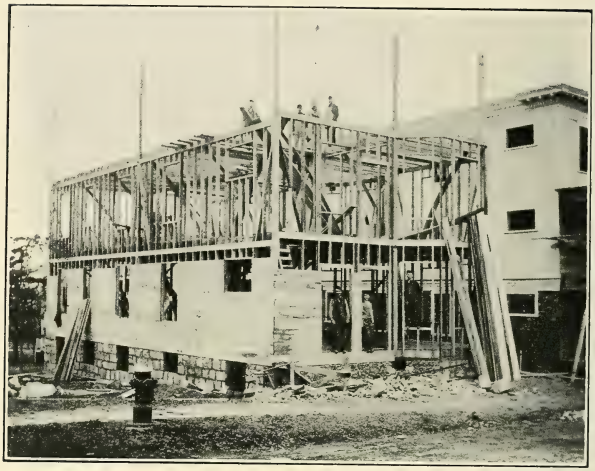

FIG. 2.—FRAME DWELLING IN COURSE OF CONSTRUCTION.

FIG. 3.—A MONOLITHIC REINFORCED CONCRETE DWELLING

Gasolene Vapor Gas Lighting Machines, Lamps and Systems 1910
Gasolene Vapor Stoves, for cooking and heating 1903
Grain Dryers 1901
Gravity Tanks 1910
Hose Couplings and Hydrant Fittings, for public fire service 1908
Hose Houses, for mill yards 1905
Incubators and Brooders 1908

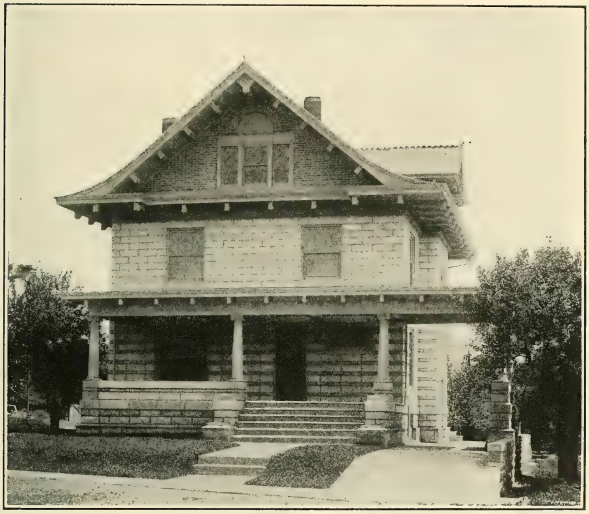

FIG. 4.—A CONCRETE BLOCK DWELLING.

Kerosene Oil Pressure Systems 1902
Lightning, Protection Against 1907
Nitro-Cellulose Films (storage and handling).. 1910
Oil Storage (fuel), storage and use of fuel oil and construction and installation of oil burning equipment 1902
Oil Storage (inflammable), systems for storing 250 gallons or less of fluids which at ordinary temperatures give off inflammable vapors 1904

Oxy-Acetylene Heating and Welding Apparatus 1910
Railway Car Houses (storage and operating), Construction and Protection of 1907
Signaling Systems, used for the transmission of signals affecting the fire hazard 1907
Skylights 1906
Sprinkler Equipment, automatic and open systems 1910
Steam Pump Governors and Auxiliary Pumps. 1908
Uniform Requirements ("slow-burning" construction, "inferior" construction, general hazards, oil rooms, general protection, stairway and elevator closures, watchmen, thermostats, etc.) 1906
Valves, Indicator Posts and Hydrants, for mill yard use 1910
Waste Cans, Ash Cans, Refuse Barrels, Fire Pails and Safety Cans for Benzine and Gasolene 1903
Wired Glass and Metal Window Frame Construction 1906
Underwriters' Laboratories, general information in reference to the nature of its work and the terms and conditions under which tests of fire appliances and materials are conducted 1909

2. Pamphlets published by The Associated Factory Mutual Fire Insurance Company, Boston, Mass.

Suggestions for Electric Lighting Equipments 1902
Specifications for Wooden Tanks and Notes on Steel Tanks 1903
Rules for Dry Pipe Systems and Automatic Sprinklers 1905
Specifications for Rotary Fire Pumps 1906
Specifications for Steam Pump Governors and Auxiliary Pumps 1907
Specifications for Centrifugal Fire Pumps..... 1907
Beltway Fires 1907
Rules for Electrical Light and Power Equipments 1907
Specifications for Fire Hose, Play Pipes and Hose Houses 1907

Specifications for Underwriters' Steam Pumps.	1907
Specifications for Valves and Hydrants	1908
Approved Fire Protection Appliances	1909
Approved Electrical Fittings	1910
Specifications for Cast Iron Pipe	1910
Approved Appliances	1910

3. Miscellaneous publications.

(*a*) Approved devices and materials tested by the Underwriters Laboratory, published quarterly by the National Fire Protection Association, Boston, Mass.

(*b*) Statistical Abstract of the United States, 1909, prepared by the Department of Commerce and Labor.

(*c*) Publications of the Spectator Company, New York, N. Y.

(*d*) Rating slips of the different rating associations of the United States.

LIST OF THE RATING ASSOCIATIONS OF THE UNITED STATES.

Association of Fire Underwriters of Baltimore City, 8 South Street, Baltimore, Md.

Board of Fire Underwriters of Allegheny County, Commonwealth Building, Pittsburgh, Pa.

Board of Fire Underwriters of the Pacific, Merchants' Exchange Building, San Francisco, Cal.

Board of Fire Underwriters of the Territory of Hawaii, 11 McIntyre Building, Honolulu, T. H.

Boston Board of Fire Underwriters, 55 Kilby Street, Boston, Mass.

Buffalo Association of Fire Underwriters, 97 Dun Building, Buffalo, N. Y.

Canadian Fire Underwriters' Association, 524 Coristine Building, Montreal, Quebec, Can.

Chicago Board of Underwriters of Chicago, 159 La Salle Street, Chicago, Ill.

Cincinnati Fire Prevention Bureau, 1403 First National Bank Building, Cincinnati, Ohio.

Cleveland Inspection Bureau, Park Building, Cleveland, Ohio.

Fire Underwriters Inspection Bureau, 1408 Merchants Exchange Building, San Francisco, Cal.

Hudson Inspection Bureau, 75 Montgomery Street, Jersey City, N. J.

Indiana Inspection Bureau, 1012 Lemcke Building, Indianapolis, Ind.

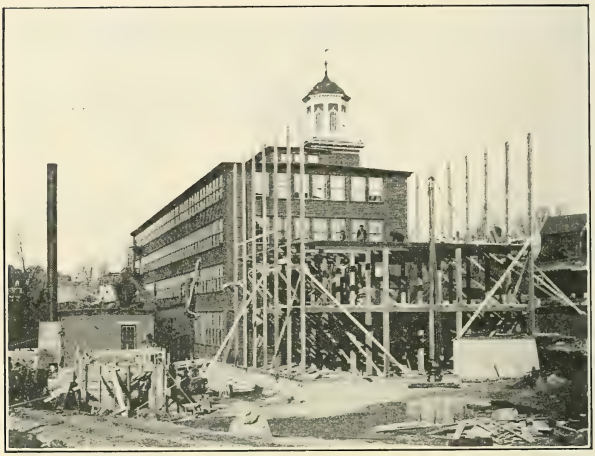

FIG. 5.—FACTORY BUILDING OF WOOD FRAME CONSTRUCTION.

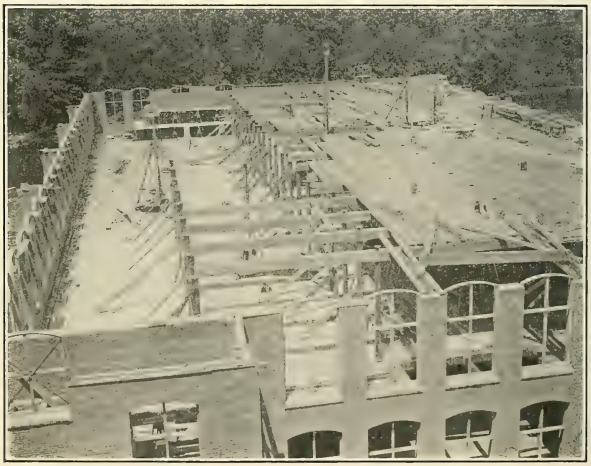

FIG. 6.—A FACTORY BUILDING OF MILL OR SLOW-BURNING CONSTRUCTION.

Insurance Association of Providence, Wilcox Building, Providence, R. I.

Kansas Fire Insurance Inspection Office, 701 Jackson Street, Topeka, Kan.

Kentucky Inspection Bureau, 105 Todd Building, Louisville, Ky.

Louisville Fire Prevention Bureau, 869 Hibernia Building, New Orleans, La.

Louisville Board of Fire Underwriters, Kenyon Building, Louisville, Ky.

Mainland Fire Underwriters' Association of British Columbia, P. O. Box 820, Vancouver, B. C.

FIG. 7.—FACTORY OF REINFORCED CONCRETE CONSTRUCTION.

Michigan Inspection Bureau, 36 Congress Street, Detroit, Mich.

Middle States Inspection Bureau, 58 William Street, New York, N. Y.

Milwaukee Board of Fire Underwriters, Mitchell Building, Milwaukee, Wis.

Minnesota and North Dakota Fire Underwriters, New York Life Building, Minneapolis, Minn.

Mississippi Inspection and Advisory Rating Company, 503 First National Bank Building, Vicksburg, Miss.

Missouri Inspection and Survey Bureau, American Bank Building, Kansas City, Mo.

Nebraska Inspection Bureau, Room 101 McCague Building, Omaha, Neb.

Newark Inspection Bureau, 756 Broad Street, Newark, N. J.

New Brunswick Board of Fire Underwriters, St. John, N. B.

New England Insurance Exchange, 141 Milk Street, Boston, Mass.

New Hampshire Board of Fire Underwriters, Concord, N. H.

New York Board of Fire Underwriters, 123 William Street, New York, N. Y.

New York Fire Insurance Exchange, 123 William Street, New York, N. Y.

Nova Scotia Board of Fire Underwriters, 121 Hollis Street, Halifax, N. S.

Ohio Inspection Bureau, Outlook Building, Columbus, Ohio.

Philadelphia Fire Underwriters' Association, 135 South Fourth Street, Philadelphia, Pa.

Philadelphia Suburban Underwriters' Association, 331 Walnut Street, Philadelphia, Pa.

Rocky Mountain Fire Underwriters' Association, 512 Exchange Building, Denver, Col.

Southeastern Underwriters' Association, Equitable Building, Atlanta, Ga.

St. Louis Fire Prevention Bureau, 734 Pierce Building, St. Louis, Mo.

Suburban Fire Insurance Exchange, 123 William Street, N. Y.

Tennessee Inspection Bureau, 1218 Stahlman Building, Nashville, Tenn.

Underwriters' Association of the Middle Department, 316 Walnut Street, Philadelphia, Pa.

Underwriters' Association of New York State, 1000 Onondaga Bank Building, Syracuse, N. Y.

Underwriters' Bureau of Middle and Southern States, 68 William Street, New York, N. Y.

Western Actuarial Bureau (Fire), 125 Monroe Street, Chicago, Ill.

Western Canada Fire Underwriters' Association, P. O. Box 3003, Winnipeg, Manitoba, Can.

West Virginia Fire Underwriters' Association, Wheeling, W. Va.

West Virginia Inspection Bureau, Schmulbach Building, Wheeling, W. Va.

Part IV.

The following information regarding changes in rates is arrived at by the study of several of the different rating slips used by the various rating associations of the country:

	Usual Reduction in Rate, Per Cent.
Automatic Sprinkler	30
Automatic Sprinkler with Watchman and Watchman's Clock	30 to 50
Automatic Fire Alarm	10
Water Proof Floors	5
Auxiliary Fire Alarm	10
Electric Supervisory System	10

Standard Automatic Fire Doors Throughout Standard Automatic Fire Windows Throughout Standard Automatic Fire Windows on exposed sides	Large variation in reduction and should be investigated by owners.

Part V.

The Committee wishes to offer the following resolution for adoption by the Association:

Whereas, The Standards of this Association include specifications for forms of concrete construction which if followed will improve the state of this art, reduce the loss of life and property by fire and generally improve the standards of building construction in America; be it

Resolved, That these facts and these Standards be called to the attention of the state and municipal building departments and the various organizations of fire underwriters in the United States, accompanied with a request that these Standards receive recognition in existing laws and in schedules for estimating insurance rates.

In conclusion the Committee submits the following statement from the president of one of the large insurance companies, setting forth the point of view of the Insurance companies regarding concrete construction which will be of interest to the Association:

ADVANTAGES OF CONCRETE CONSTRUCTION.

City risks not ordinarily open to competitive rates on account of outside exposures may be brought into the competitive class by this method of construction, which will enable the property to be thoroughly protected against outside exposures.

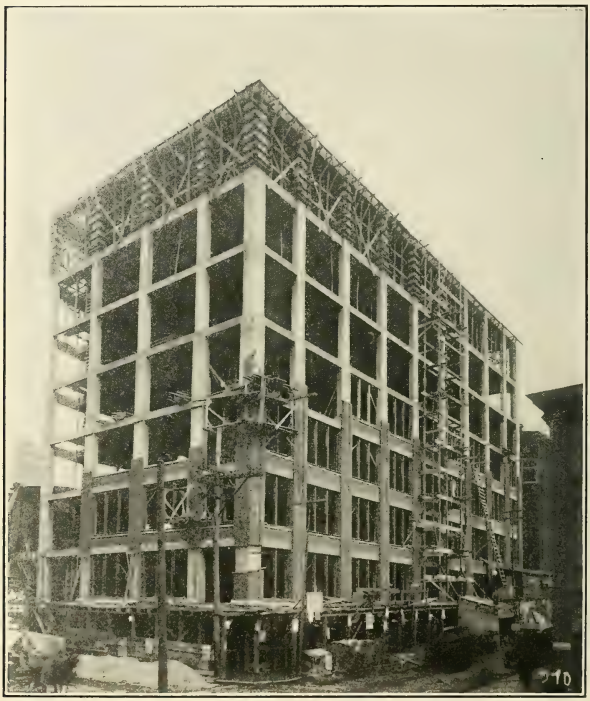

FIG. 8.—A MERCANTILE BUILDING OF REINFORCED CONCRETE CONSTRUCTION.

Under this form of construction it is practicable to have all the manufacturing processes and the storage under one roof, making the factory self-contained and more economical in operation. On account of superior construction special hazards can be thoroughly cut off and

protected. The storage of goods or material specially subject to water damage can be rendered practically immune, as floors can be made practically water tight.

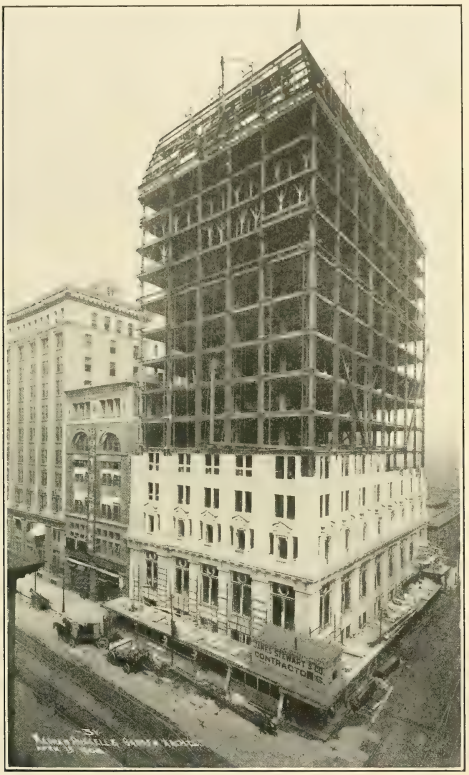

FIG. 9.—A MERCANTILE BUILDING OF STRUCTURAL STEEL CONSTRUCTION, FIREPROOFED.

Owing to the superior construction, many classes of risks may be placed in the competitive class that would not be considered under other types of construction.

This method of construction results in a decreased cost of the sprinkler equipment, as it is necessary to protect only the contents of the building and not the building itself.

In buildings of this character with a non-combustible occupancy sprinkler equipment can be entirely eliminated, thereby saving the expense of at least 5 cents per square foot of the entire floor area.

Respectfully submitted,

W. H. HAM, *Chairman.*
EMILE G. PERROT,
F. W. MOSES,
W. H. MERRILL,
J. P. H. PERRY.

AN INCIDENT OF THE VALUE OF CONCRETE IN REDUCING THE COST OF INSURANCE.

By Emile G. Perrot.*

The large steel sprinkler tank and tower of the Victor Talking Machine Company, Camden, N. J., is an instance of the beneficial result derived from the use of concrete from an insurance standpoint.

This tank holds 100,000 gallons of water, the bottom of which is 125 ft. from the ground. The total height of tank and tower is 174 ft. The diameter of the tank is 20 ft. The tower is erected on filled ground near a river and the foundations of the four legs are supported on concrete piles.

By reason of the close proximity of buildings belonging to other owners, the Victor Talking Machine Company desired to take out indemnity insurance on the structure as originally designed, that is, without concrete protection, so that in case of a fire in the buildings adjacent and the collapse of the tower, the damage to adjoining property would be covered. This, they were informed, would cost $500 per annum. It was finally decided not to place insurance on the structure, but instead to invest about $4,000 in fireproofing the tower with concrete, as shown in Fig. 1. A little reflection will show that the investment fully pays for itself, not only financially, but by practically making the tower immune from fire and the destructive agencies of the atmosphere, which it would not be if left unprotected. Charging against the tower the interest on $4,000 at 6 per cent. per annum, gives $240 as the carrying charge for the absolute protection against fire, instead of a $500 charge for no absolute protection to person or property, but simply to cover damages for which the company might be liable in case of a collapse of the tower.

This clearly shows how it pays to carry your own insurance, so to speak, by taking precautions to avoid damage to persons

* Ballinger and Perrot, Architects, Philadelphia, Pa.

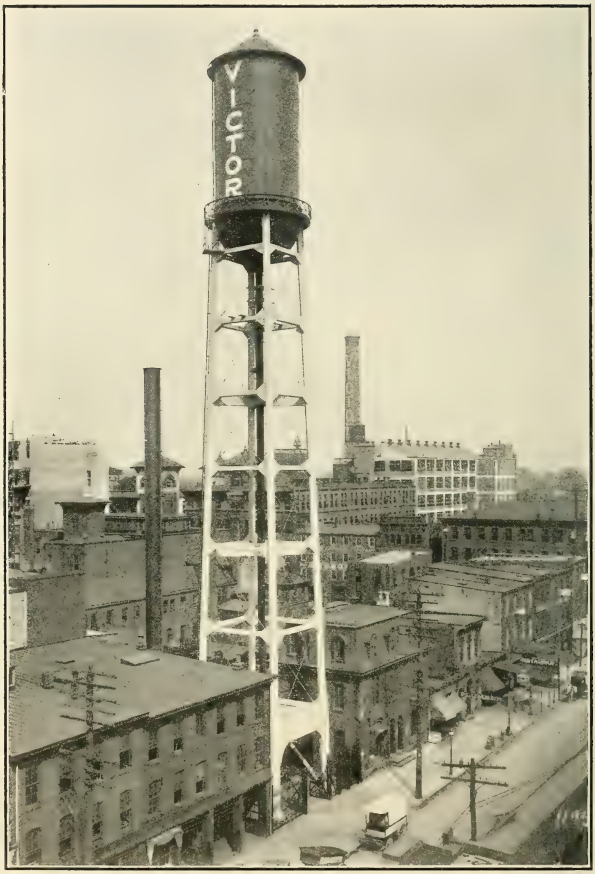

FIG. 1.—STRUCTURAL STEEL WATER TOWER FIREPROOFED WITH CONCRETE. VICTOR TALKING MACHINE COMPANY, CAMDEN, N. J.

or property by making a building fireproof. Further, the saving on the insurance premium will pay for the fireproofing in about 15 years, so that at the end of that time the original cost of the fireproofing will have been wiped out. This, of course, is based on the assumption that insurance would be carried on the tower if it were not fireproofed.

The details of the fireproofing are as follows: The thickness of the concrete outside of the furthest projection of all members is 3 ins.; the main legs of the tower are reinforced with four vertical 1-in. square twisted bars, tied together with 3/16-in. wire ties every 9 ins. Sheet iron clips are fastened to the corner of the structural legs, so as to register the reinforcement in its proper position.

On account of the difficulty of reinforcing the diagonal rods of the tower, they are not fireproofed, and in order to make the tower rigid in case of fire heavy reinforced concrete gussets were put in the angles between the vertical legs and the horizontal struts, reinforced with two ¾-in. square twisted bars. The thought being that the reinforced concrete gussets would be sufficiently rigid to prevent the tower from distorting should the rods become heated in a fire.

Close inspection has lately been made of the fireproofing, and there is no evidence of any cracks in the concrete. This is remarkable on account of the extreme height of the tower and the fact that the tower has been subject to the extreme temperatures of both winter and summer. However, there has been an abundance of reinforcement placed in the fireproofing to prevent cracks from thermal stresses.

The mixture of the concrete is one of cement, two of sand and four of stone, ½-in. broken trap rock being used.

THE HENRY HUDSON MEMORIAL BRIDGE.

By William H. Burr.*

The Henry Hudson Memorial Bridge has not yet been built, but it has been completely designed in all its features, and the work is practically ready to be advertised and awarded as a contract. The great arch which forms the main feature of the bridge has been designed, worked out in all its details and is absolutely complete in plans and specifications. In fact, I suppose there is standing scarcely a structure either in masonry or metal that has been so minutely studied or carefully considered in every feature and detail from foundation to false work as the Henry Hudson Memorial Bridge. I shall not attempt to go into great detail, but shall endeavor to touch upon the main features of the design about which there may be some interest from an engineering point of view.

The total structure is about 2,900 ft. in length and it crosses Spuyten Duyvil Creek about 800 ft. east of the New York Central and Hudson River Railroad station of that name, from the rocky highland on the southerly side of the creek to the high ground northerly of it where fortunately the rock is near the surface. The great arch thus springs from rock near the surface on one side to rock at the surface on the other, thus eliminating all real foundation problems. This circumstance, together with the additional feature of a sufficiently high rise, makes it easily feasible to construct the arch, which would not be practicable if the roadway had to be at much less elevation. The approaches are designed to be constructed of the native stone on either side of the river and such as is found on many parts of Manhattan Island. It is the native gneiss, much of which is building stone of excellent quality. Nothing further will be said in regard to the approaches, although they are well worthy of much favorable comment. It is necessary to devote the short time at our disposal to the main arch.

*Consulting Engineer, New York, N. Y.

This arch has a clear span of 703 ft., and the soffit at the crown is about 184 ft. above the water of the Spuyten Duyvil. It is designed to carry on its upper deck a roadway 50 ft. wide and two 15-ft. sidewalks, making a deck 80 ft. wide. About 18 ft. below the roadway is a deck on which there will be placed four railway tracks for rapid transit lines, running up into the northern part of the city. It is therefore not altogether a memorial, but a part of the rapid transit subway system of this city, yet so designed and constructed with suitable features as to be in the best sense a memorial.

The rib of this great arch, having a span from center to center of springing joints of 725 ft., has a width of 70 ft. and its depth at the crown is 15 ft.; that is, a solid rib 70×15 ft. in cross section at the crown and 70×28 ft. at the springing joints. It is interesting to observe in passing that the investigations made to determine the feasibility of this arch, which had been pronounced not feasible by a number of engineers, showed that the length of span with corresponding rise might be carried above a thousand feet. Thus this great length of 703 ft. clear span is not by any means the extreme limit.

The reinforcement, some features of which will be shown, is so formed and designed as to constitute a load carrying steel structure. That is, the percentage of cross-section of steel is rather high, although not excessively so. It varies from about 1½ per cent. at the crown to a little over 3 per cent. at the springing. This steel reinforcement consists of 48 units, each unit being composed of four 8-in. steel angles placed at the corners of a rectangle 2 ft. 10 ins. wide and about 3 ft. high. There are 4 such units in each vertical plane and there are 12 vertical groups of 4 each, making 48 altogether. Each unit of four angles is held together by latticing on the four sides, and there is suitable latticing between adjoining units. Hence this steel reinforcement is held rigidly together as a carrying member, and there are distinct advantages in such an arrangement.

The haunches of the arch, or the spandrels, are made of arch openings of the usual character, the arches being supported upon reinforced concrete piers rising from the main rib. There again the steel reinforcement is of the load carrying character, so that it

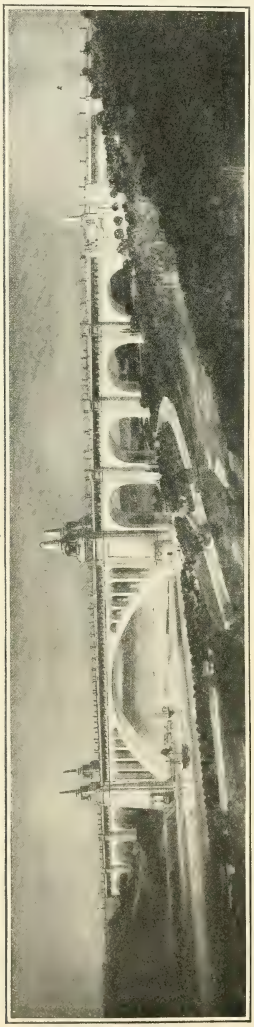

FIG. 1.—THE PROPOSED HENRY HUDSON MEMORIAL BRIDGE.

not only reinforces the concrete, but it also carries load as a column. The resultant effect in the main rib and in these spandrel piers and arches is to secure, after the concrete is put in place, a reinforced mass in which the steel reinforcement is so interlaced and bound together as to make the combination of concrete and steel act absolutely as a unit. There is no possibility of any part of the steel or the concrete acting by itself.

Probably no reinforced concrete work has yet been designed or executed in which these results are so completely accomplished as in this particular structure. The consequence is that the concrete may be subjected to a materially higher working stress than would otherwise be permissible. In this design the nominal upper limit of the working pressure in the concrete was taken at 750 lbs. per sq. in. As a matter of fact when the design was complete it was found that the working pressure, including all the direct stresses of the load and the stresses due to change of temperature and change of form when the false work is removed, did not exceed 700 lbs. per sq. in. If the ordinary methods of computation had been employed, that is, those ordinarily used in connection with reinforced concrete arches, the working pressure would have been not over 600 lbs. per sq. in.; but the secondary or indirect stresses of temperature changes and changes of form added the difference. Furthermore, instead of assuming that the greatest stresses would exist with one-half of the span loaded, or five-eighths more or less, the analysis was so worked out as to make it possible to load such portions of the span as would give maximum stresses at all points.

It is my judgment that it would be entirely proper to use a materially greater intensity of stress than 700 or 750 lbs. per sq. in. The dead load is so large compared with the live load that probably even 1,000 lbs. per sq. in. would be a reasonable working stress to be employed. It would scarcely be advisable to use such a high working stress in ordinary structures of this class, but I believe it would be entirely safe and proper in this case.

The problem of putting the material in place on the false works has also been most carefully worked out. Without attempting to go into this matter to any great extent, it may be interesting to say that the false works, which have been designed with just as much care as the structure itself, will be supported by piles with an opening 100 ft. wide to accommodate navigation. The

false works will be of timber except the truss spanning the opening over the railroad track near the northern abutment; and it will probably be found advisable to keep the timber work constantly wet, not only as an insurance against fire, but also to prevent the shortening of the timber members by drying and shrinkage.

Before going on to further discussion in detail it may perhaps be advisable to take a view first of the general structure and of certain main features of the steel work, after which will follow the discussion of other features of the work and some matters of interest in connection with the general subject of such masses of reinforced concrete.

The view in Fig. 1 shows the bridge as a whole. The approaches to the arches on either side are those portions of the structure which I have said would be built of native stone. The main arch, 703 ft. clear span, is shown in the center. The spandrel arches are seen at each end. The main arch is reinforced by 48 units, and having its soffit 184 ft. above the water with an upper deck about 217 ft. above it. I think you will all agree with me that such a structure would be a credit to the metropolitan city of this country. It would be a great public work, not wholly memorial, but more useful even than memorial; with its ornate character emphasized rather in the impressive dimensions of the structural whole than by any special ornamental work put upon it.

The upper deck, Fig. 2, Plate I, has a 50-ft. roadway and two 15-ft. sidewalks, and 18 ft. below it are seen the four rapid transit railway tracks. The extreme width of the structure, from out to out, is 85 ft. The width of the rib on which the decks are supported is 70 ft. The steel lines are drawn so heavy in reference to the outer lines of the concrete that the steel appears to be the principal part of the structure instead of being a very subordinate part. It is probable that the four railroad tracks would not all be put in place at the time of the completion of the bridge, although two tracks would probably be so employed at once.

The main feature, which is evident from an inspection of the views, is the fact already emphasized that the steel work is not only a reinforcement of the concrete, but also a load carrying element of the construction.

Fig. 3, Plate I, shows on the right an elevation of one of the

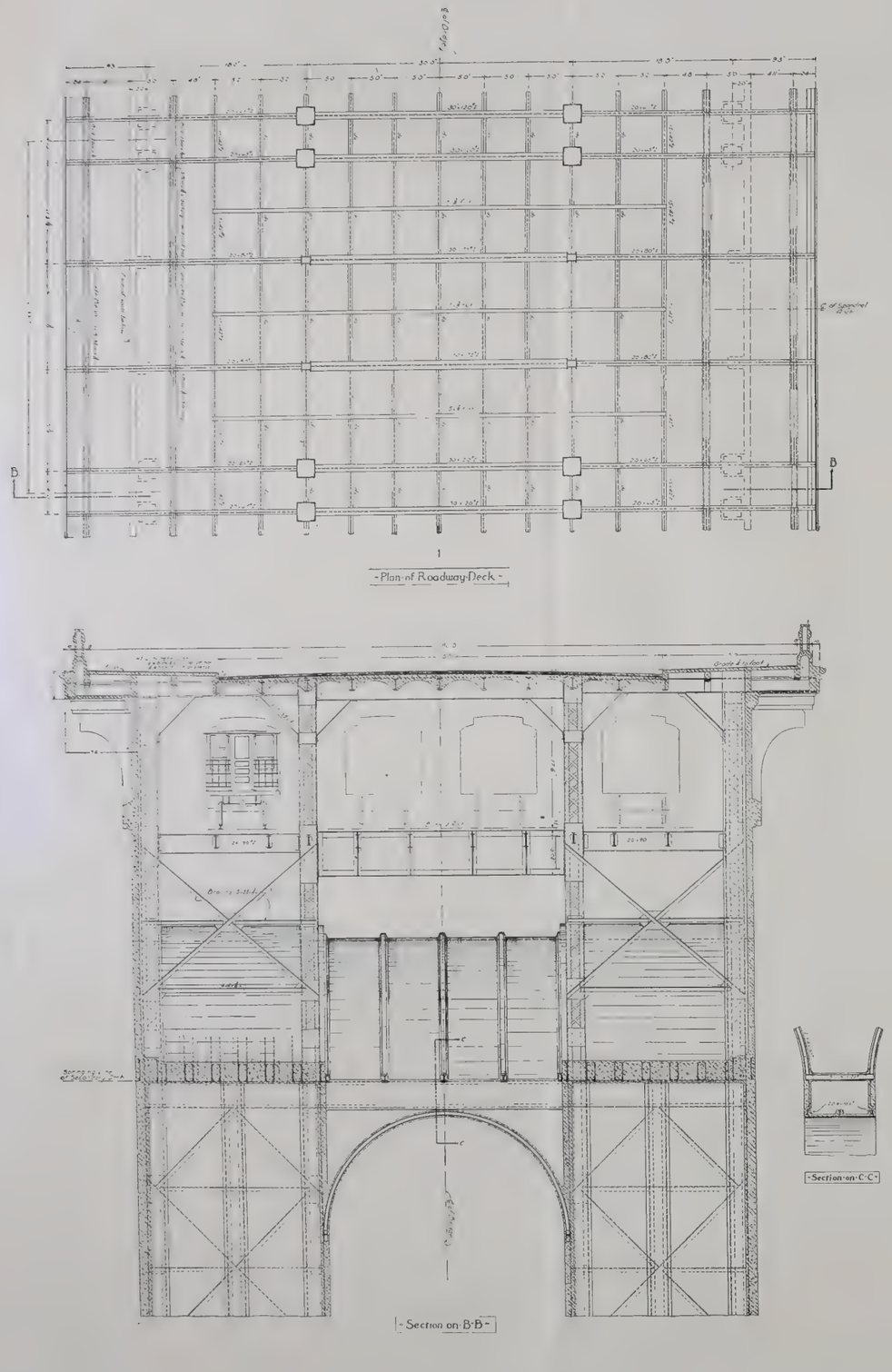

FIG. 2.—CROSS-SECTION THROUGH SPANDREL PIERS.

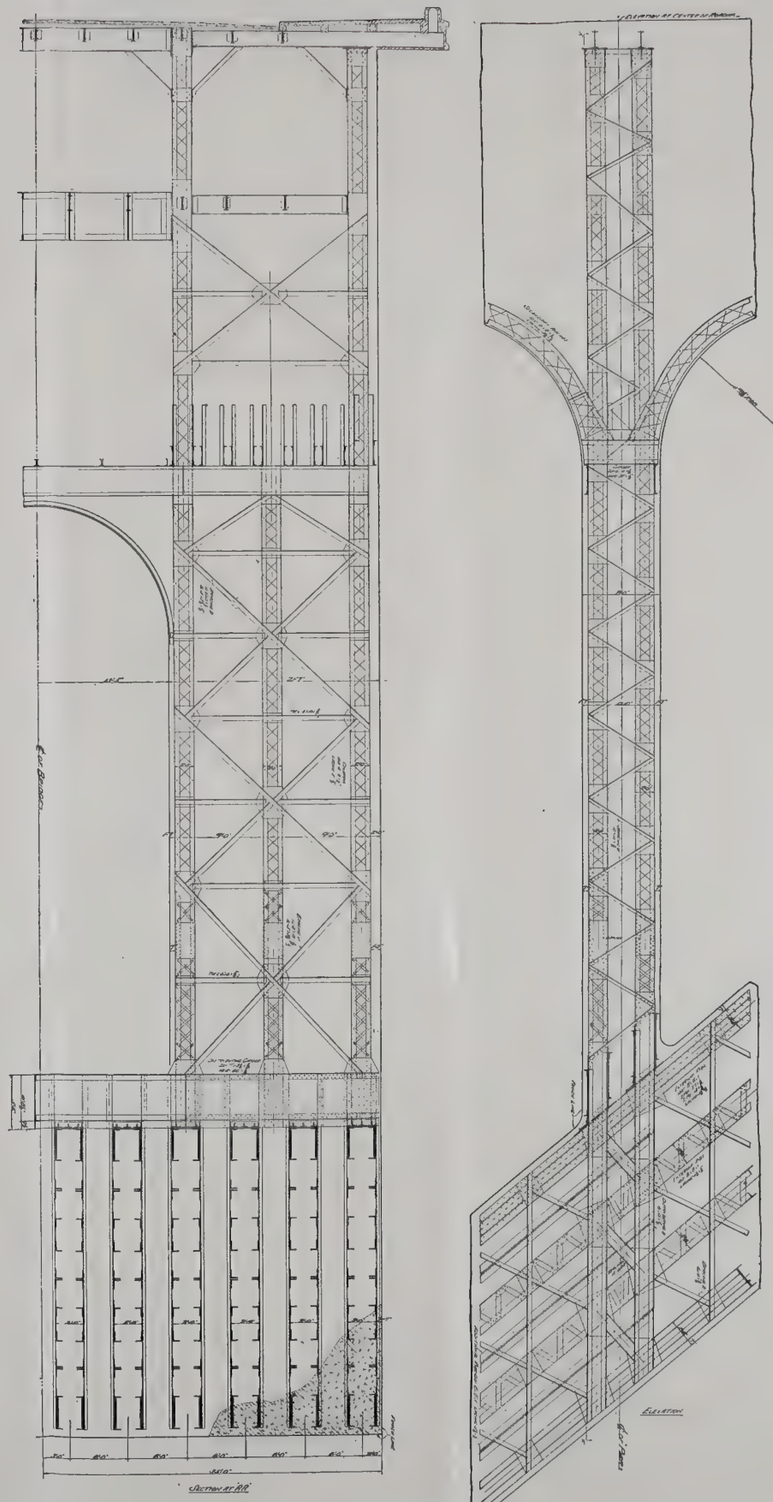

FIG. 3.—SECTION AND ELEVATION OF PIER 2.

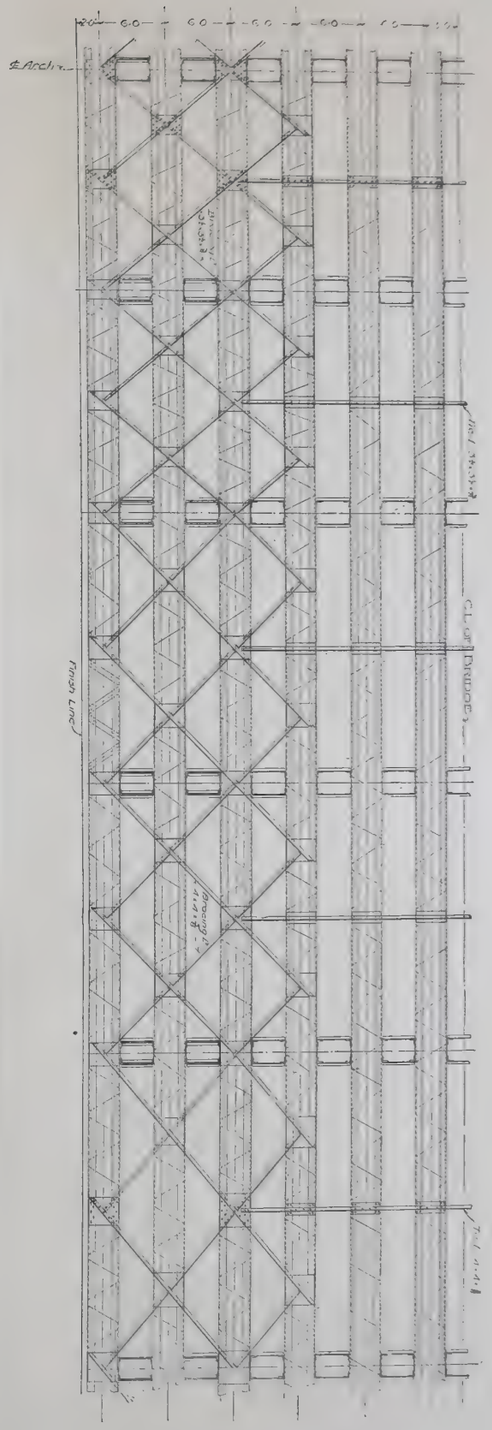

FIG. 4.—PLAN SHOWING LATERAL BRACING OF RIBS.

spandrel arches, with the comparatively light steel posts, which form the reinforcement, and below them the four tiers of the units extending throughout the length of the arch and the intermediate light steel reinforcing ribs which begin down near the springing joint. These steel ribs with the bracing between them in their upper and lower surfaces, concentric with the soffit of the finished arch, complete the steel reinforcement of the main rib. It is evident that this reinforcement is so distributed and interlaced throughout the entire mass as to constitute an inseparable unit of the whole. The concrete thus acquires a materially enhanced capacity to carry load through a system of load carrying steel reinforcement. Fig. 4, Plate I, shows the lateral bracing of the ribs.

It is obvious in order to make a satisfactory structure that the concrete must be a relatively wet mixture, which I think is the better practice for any concrete structure. The outside forms of this great rib must be continuous and probably of wood, although they might be lined with some metal such as steel. It is necessary that the timber forms should have smooth surfaces and be held accurately in position and line throughout the operation of depositing the concrete. The concrete must be wet enough to fill the interior of these 48 reinforcing units and all the spaces between them. It will be necessary, of course, to resort to the usual spading or agitation of the concrete, so that as nearly as possible all air bubbles may be removed and also that coarser parts of the aggregate may be kept from contact with the outside forms, so that on the removal of the timber forms a reasonably smooth concrete surface may be exposed.

There are evidently some serious problems in connection with the depositing of the concrete in such a structure. In the first place there are some details of that work which it is practically impossible to outline prior to the beginning of the deposition of the concrete. The plan in the main is easily laid down. The concrete will undoubtedly be placed in the forms in voussoirs, probably of the full depth, but not necessarily so. Furthermore, those voussoirs would be so located throughout the length of the rib as to reduce the initial stresses in the completed mass to a minimum.

The ratio between the coefficients of elasticity for steel and concrete is taken at 15. That being the case, 700 lbs. per sq. in.

being the limit of pressure in the concrete, obviously the stresses in the steel would be fifteen times that or about 11,000 lbs. per sq. in., which is too low. It is therefore of consequence to resort to some method to impress upon the steel an initial stress, so that when the completed mass is in position that steel shall have a higher stress than 11,000 lbs. per sq. in.; and that can easily be accomplished. It is intended that the steel work, as I have already intimated, shall be erected as a light structure in itself, and that after it is so erected in place, before any concrete is deposited, it shall be relieved from any support by the false work, so that it will carry its own weight. If found desirable it may be loaded in addition to its own weight. It is intended in this way to accomplish such addition of initial stress before the concrete is put in place as to give to it a final stress of about 20,000 lbs. per sq. in. This procedure has the advantage not only of compelling the steel to take a greater proportion of load than it would otherwise carry, but, also, of relieving the false work of the weight of the steel.

One or two interesting questions arise at this point. First, that of forming the voussoirs with the reinforcing steel in place. It is intended to form the voussoirs by means of wire netting; that is, the voussoir joints will be made by sheets of wire netting, secured by wire to the reinforcing steel ribs. That can be economically and quickly done and there is at least saved the labor of removal of forms. The wire netting is left imbedded in the concrete and remains as a permanent part of the structure. What the size of mesh shall be is perhaps not capable of exact expression at the present time, but it would appear from some experiments which have been made by the Department of Bridges in connection with this work, that from ½ to ¾ of an inch may be about right. Some of the finer part of the wet concrete would find its way through the meshes, but a very small part only. It is a matter of no consequence whether the wire netting joint is kept plane or not, or whether it bulges by the pressure of the concrete. In fact, it is a matter of some advantage not to have it plane, because when the next voussoir is put in place any irregularity of the voussoir surface or joint will be productive of a better bond with the adjoining new concrete. Similarly any roughness of surface which would be produced by depositing the concrete will improve

the bond between the new concrete and that already in place. By using such a method as that contemplated there will be but the exterior timber forms to be removed, including the timber form for the soffit of the arch.

Another point which will probably suggest itself to many engineers is the condition of the ends of the rib. It produces simpler stress conditions to construct an arch rib with hinged ends, but it does not make so stiff a structure. It is essential in such a case as this to make the rib as nearly perfectly continuous as possible with actually fixed ends. To do this the reinforcing steel of these ribs will be carried down into the rock and anchored there, so that the ends will be as rigidly fixed as it is possible to make them. Evidently it is not possible to make the ends of the rib mathematically fixed, but it is possible by proper means to give the ends such a rigid attachment to the rock as to make them for all practical purposes fixed, so that the assumption of fixed ends does not involve any material error.

The subject of the final surface finish is one of great importance in such a structure because of its ornate character. I think that every one who considers such a great structure in which the proportions are really the elements of its effective appearance will reach the conclusion that there should be as little superficial ornamentation as possible. The Department of Bridges was fortunate in this case to secure the services of Mr. Whitney Warren as the consulting architect, and the architectural features of this entire structure have been worked out under his direction and supervision. He wisely concluded to treat this great concrete mass honestly as such a mass. The subject of paneling the soffit arose early in the architect's studies and it was at first thought that it would be best to make a suitable paneling of the soffit, but the consulting architect finally concluded to omit such superficial markings and let the soffit stand as a great plain concrete surface, and the sides of the ribs were treated in the same way. It has been his plan to permit the concrete surface to stand in its own honesty and simplicity; and I believe that when the structure is finished the decision of that question will be justified by the impressive appearance of a great structure of perfect simplicity but of such dimensions as to give it the quality of grandeur.

It will be a matter of interest to state what the cost of such a structure will be and approximately to indicate its relative cost in comparison with that of a steel arch bridge. It is estimated liberally that the entire structure, with its great capacity for moving load amounting to 15,000 lbs. per lineal foot of structure, will cost about $3,800,000. Under a proper execution of the work the actual cost is likely to be less rather than more than the above amount.

It has been carelessly stated even in some engineering papers that the concrete of this particular structure is carried by the steel. That is not only untrue, but very far from the truth. The concrete not only carries itself but its own proper part of the moving load. In other words, even if the concrete were unreinforced it would not only carry its own weight but much live load besides. The advantage of the system of reinforcement adopted lies in the fact that it not only reinforces the concrete, thus enabling it to carry much more load than would otherwise be feasible, but it also sustains not only its own weight but its due share of the moving load. The span and rise have such relation that the concrete is economically used.

The matter of expansion and contraction in both plain and reinforced concrete structures is of marked importance. Obviously no temperature joints can be introduced into an arched rib, but there are parts of the structure, such as the concrete floor plates, in which such joints should be introduced. As essential as these joints are in their proper place, the provision of expansion joints in both plain and reinforced concrete work has been much exaggerated, just as it was in connection with steel structures in the early days of steel work. A reinforced concrete arch structure a thousand feet long has been constructed and has stood the successful endurance of atmospheric changes without a single temperature provision in it from one end of the structure to the other, except in the concrete floor plates and one or two other similar parts.

CONCRETE FILLED STEEL ARCHES.

By Henry H. Quimby.*

The bridge carrying Forty-second Street over the Main Line of the Pennsylvania Railroad in Philadelphia is believed to be the first example of a new type of construction that will be found by designers to be economical and applicable in a number of cases.

The bridge is a pure unbraced arch with suspended floor, the arch being in two ribs of steel plate box construction of rectangular section 15¼ ins. inside width and varying depth, filled with concrete. The floor is of structural steel and concrete, built girder cross floorbeams with cantilever extensions for outside sidewalks, and under driveway floor rolled beam stringers with concrete jack arches, the sidewalks being rod-reinforced concrete slabs. Fig. 1 shows a side view of the finished structure; Fig. 2 an end view.

The arch ribs are not hinged and yet are not quite hingeless. They are fixed—or rather square ended—at the spring, and at the crown, though 3 ft. deep, are keyed with a steel plate that gives a bearing depth of only 16 ins. The depth of the arch ring at the spring is about 10 ft. and at the crown 3 ft. abruptly reduced to 16 ins. on the key. This "semi-hingeless" construction was adopted for the purpose of obtaining stiffness against variable loads without excessive depth at the quarters and without inducing very high temperature stresses. If hinges had been used at the spring the arch would have required to be made much deeper at the quarters than at the ends, giving a very ungraceful appearance, and besides, the construction would have cost considerably more. Hinges practically eliminate temperature stresses and enable us to determine more accurately the line of pressure for any given condition of loading, but in arches with good pitch—ratio of rise to span—the temperature stresses are not so severe as in flat arches, and therefore the advantages of hinges do not in such cases compensate for the increased cost and awkward appearance that they entail.

The economy of the arch type of bridge in this case is found

*Engineer of Bridges, Bureau of Surveys, Philadelphia, Pa.

in dispensing with a tension chord and a web shear system, the excess of the arch section at the spring over a compression truss chord being slight in comparison, and the greater cost of the masonry being also considerably less than the saving in steel. In fact, the total cost of the masonry as built in this case was less than that of tension chords would have been for a truss span, though it had to be carried down about 40 ft. to get satisfactory bottom for foundations.

FIG. 1.—FORTY-SECOND STREET BRIDGE OVER PENNSYLVANIA RAILROAD, PHILADELPHIA, PA.

The office of the concrete filling of the arch ribs is threefold. It takes the place of interior steel diaphragms, an extensive system of which would be required to make the wide web plates available throughout as section to carry compressive stress and to prevent buckling and distortion, and which would have cost just about as much as the concrete did. It also protects the interior surface of the box against corrosion more permanently and more conveniently than would painting, for means of future access to the interior would otherwise have to be provided, and even then the work of

repainting would be difficult, and because the need of it would not be obtrusively conspicuous it would be liable to be neglected. The concrete also gives to the arch very considerable strength for carrying compressive stress.

The floor suspenders are rolled steel I-beams (15 ins., 60 lbs.), with webs transversely, the advantages of this shape being that the suspenders could be used as posts for temporary support of the arch segments in erection, and they are competent to transmit to

FIG. 2.—END VIEW OF FORTY-SECOND STREET BRIDGE.

the rigid floor one-half of the wind loads on the arch. They also make very sightly, clean-limbed truss members, no lacing or other trimming being needed.

Elevation and plan of the bridge are given in Fig. 3, while Fig. 4 shows several details.

The arch ribs are carried upon independent concrete abutment piers, the railroad bank being sloped between them, and the floor of the bridge extended back to the crest of the bank. It was found to be more economical to span the slope with the floor than to construct a retaining wall and abutment wings and fill the space over the slope. It was also desirable to keep all foundation ex-

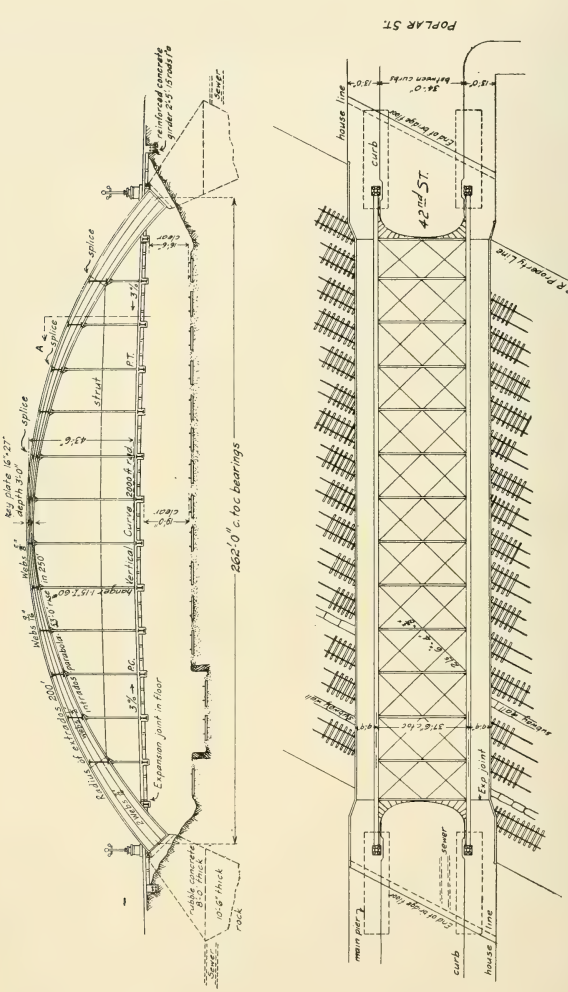

FIG. 3.—PLAN AND ELEVATION, FORTY-SECOND STREET BRIDGE.

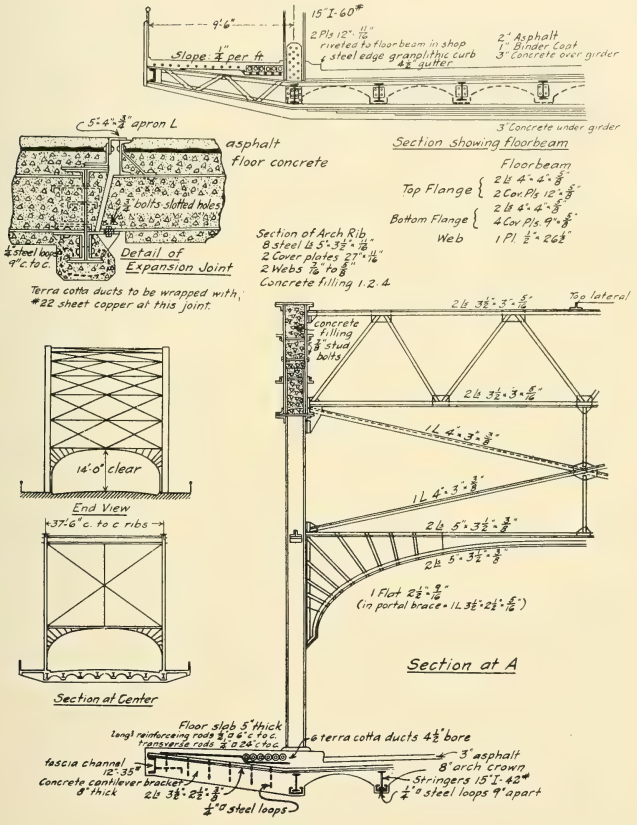
9'-6"
15"I-60#
2 Pls 12" × 1/16" riveted to floorbeam in shop
steel edge granolithic curb & gutter
Slope 1" per ft.
2" Asphalt
1" Binder Coat
3" Concrete over girder
3" Concrete under girder
5'-4" × 3/8" apron L
asphalt
floor concrete
3/4" bolts slotted holes
3/4" steel loops 9" c. to c.
Detail of Expansion Joint
Terra cotta ducts to be wrapped with #22 sheet copper at this joint.
Section showing floorbeam
Floorbeam
Top Flange { 2 Lb 4" × 4" × 5/8"
2 Cov. Pls 12" × 3/8"
Bottom Flange { 2 Lb 4" × 4" × 5/8"
4 Cov. Pls. 9" × 3/8"
Web 1 Pl 1/2" × 26 1/2"
Section of Arch Rib
8 steel Lb 5" × 3 1/2" × 7/16"
2 Cover plates 27" × 1/16"
2 Webs 7/16" to 5/8"
Concrete filling 1:2:4
Top lateral
2 Lb 3 1/2" × 3" × 5/16"
concrete filling
3/4" stud bolts
2 Lb 3 1/2" × 3" × 5/16"
1 L 4" × 3" × 3/8"
1 L 4" × 3" × 3/8"
2 Lb 5" × 3 1/2" × 3/8"
2 Lb 5" × 3 1/2" × 3/8"
1 Flat 2 1/2" × 9/16"
(in portal brace = 1 L 3 1/2" × 2 1/2" × 5/16")
14'-0" clear
End View
37'-6" c. to c ribs
Section at Center
Section at A
Floor slab 5" thick
Longt reinforcing rods 1/2" @ 6" c. to c.
transverse rods 1/2" @ 24" c. to c.
6 terra cotta ducts 4 1/2" bore
3" asphalt
8" arch crown
Stringers 15"I-42#
1" ø steel loops 9" apart
fascia channel 12"-35#
Concrete cantilever bracket 8" thick
2 Lb 3 1/2" × 2 1/2" × 5/16"
1" ø steel loops

FIG. 4.—DETAILS, FORTY-SECOND STREET BRIDGE.

cavation away from the houses which, at the four corners of the bridge, were standing at the street line, and whose foundations were in two cases on filled ground, and in all cases far less in depth than was required for the pier foundations. The floor over the slope is similar to that suspended from the arch ribs—steel floor beams and stringers with concrete jack arches—except that the diagonal end floor beam which was built directly on the ground is a rod-reinforced concrete beam.

The abutment piers are of rubble concrete founded on a good grade of micaceous rotten rock, the pressure being about 4 tons per sq. ft. The piers are 10 ft. wide at the bottom and 8 ft. at the top, with a transverse grillage of twelve steel beams, 20 ins., 80 lb., 7 ft. long, in two groups of six each, and two additional beams at the middle to take the bearing of the temporary erection blocking.

The span of the arch is 262 ft. c. to c. of bearings on the grillage skewbacks, crossing fifteen railroad tracks at an angle of 28 deg. The rise to the centre of the rib at the crown is 52 ft. The intrados line is a parabola of 252 ft. span and 54 ft. rise. The extrados line is a segmental curve of 200 ft. radius between points 11 ft. above the floor and tangents from there to the skewbacks, which are at an angle of 45 deg. The ribs are 37 ft. 6 ins. apart c. to c., giving a driveway, which is asphalt paved, 34 ft. wide between curbs. The sidewalks are granolithic, 11 ft. 3 ins. wide from curb to centre of railing, the overhang from centre of an arch rib being 9 ft. 6 ins.

The floor with the paving forms the greater part of the load. The horizontal thrust in each rib from the steel work alone was 117 tons, and the resultant delivered to the abutments 150 tons. With the concrete filling in the arch the thrusts were 185 and 255 tons respectively. With all dead load they are 614 and 800, and with a distributed live load of 100 lbs. per sq. ft. over driveway and sidewalks 857 and 1110 tons.

The stresses produced by the steel work and arch filling together constitute the initial stress on the steel ribs. All subsequent loading must be borne by the steel and concrete jointly, because time was given the filling to set hard before any additional load was placed. As the crown keys were well fitted to an even bearing and the bearing at the spring was evenly keyed up with the steel

weight alone, the line of pressure passed very close to the middle at both points and the initial stress on the steel was therefore fairly uniform at about 3000 lbs. per sq. in. The distribution of subsequent loading between the steel shell and the concrete core must be in direct proportion to the moduli of elasticity of the two materials, whatever they may actually be, multiplied by the area of available cross-section, for neither material can shorten under stress without, or any more than, the other. If the ratio between the moduli be 15 then at maximum load the total stress on the steel, including 3000 initial, will be 11,000 lbs. per sq. in., and on the concrete 560 lbs. The concrete core itself produces a thrust that causes 1085 lbs. of the 3000 initial on the steel, or would, if it carried itself, cause a unit stress of 240 lbs. on the concrete alone, showing that the core has a carrying capacity far beyond its own dead load.

The web plates are tied together at intervals of about 2 ft. with 7/8-in. stud bolts that resisted the hydrostatic pressure of the soft concrete during casting, and serve to prevent the plates from buckling away from the concrete in compression. These bolts also afforded needed and convenient roosting places for the man who spaded the concrete as the filling proceeded and for the inspector who supervised the operation. The clear space between webs was 15¼ ins. to permit free insertion of the 15-in. beam hangers, and it afforded just about enough room for working.

The filling was accomplished by means of sheet iron spouts through 5-in. round holes in the top cover plate at intervals of about 8 ft. As the filling proceeded these holes were closed with hexagonal plates tap-bolted to the cover. Near the crown the top cover plate was sprung up and propped open enough to admit the men to the interior until the filling reached the opening when it was permanently closed and riveted. The spading to the crown was then done through the holes with rods, and the filling was finished with grout using sink-head hoppers on tap bolt holes at the highest point. The filling of the opposite halves of a rib was carried forward simultaneosuly to avoid unbalanced and distorting stresses.

The erection of the steel work was by means of partial falsework below the floor and two long boom stiff-leg derricks on it. Each rib was shipped in eight sections, and each section—except

the two middle ones—when placed in position was propped on the 15-in. beam hangers as posts. The ribs were started on narrow bearings on the skewbacks that acted as temporary hinges to facilitate control of the camber until the connection was made at the crown, which was accomplished by keeping the falsework wedging high and lowering it as needed. This temporary bearing, which was at the mid line of the arch ring, was composed of two 7 in. x 1¼ in. cast zinc blocks side by side on the two middle beams of the grillage. Zinc was adopted as the material of the temporary blocks because it will squash out under about 3000 lbs. pressure and would thus prevent excessive stress coming on the middle of the arch ring due to the initial load being carried there.

After the steel work was all placed a time of mean atmospheric temperature was awaited for the final keying at the skewback in order to divide the temperature stresses evenly between winter and summer and thus minimize the extreme. It was found by trial stress diagrams of the arch with various distributions of dead loading that with all the steel work in place, but no concrete either in arches or floor the line of pressure passed through the middle of the arch ring at the spring. This was then the proper stage of the work for fixing the bearing on the skewback.

A temperature of from 55 to 58 deg. Fahr. was found very early one morning and the bearing wedges were then driven up firmly with steel mauls. These wedges were 7 x 30 ins., tapering in thickness from ⅜ to ⅝ ins., one pair on each of the twelve grillage beams in each skewback. The aim in driving was to make them all equally tight—not necessarily to take up all the then load, but it was found that such wedging as was done released the pressure on some of the blocks—150 tons. Then holes were drilled through the wedges and tapped into the grillage beams, and tap bolts were screwed in within ⅛-in. of the heads and so left, the purpose being to avoid any possible lift or anchorage effect on the grillage if temperature or other force should open the skewback joint at either intrados or extrados.

Careful examination since in both extremes of heat and cold has failed to discover loosening of any of the wedges, though a difference in pressure on them was perceptible by sounding with a hammer. This indicates that the line of pressure has not yet wandered to either edge of the middle third of the bearing.

The live load for which the bridge was designed is a general load of 100 lbs. per sq. ft. over the whole floor, or a concentration of 110 lbs. per sq. ft. on sidewalk members, and of 40 tons on two truck axles 20 ft. apart, or two lines of street railway cars, and various combinations of these loads to find maximum possible stresses.

Each arch rib is composed of two web plates 15¼ ins. apart in the clear, varying in thickness from 7/16 ins. at the lower section to ⅝ ins. at the crown section, four flange angles and four intermediate angles 5 x 3½ x 11/16 ins., and one top and one bottom cover plate 27 x 11/16 ins. The intermediate angles are counted in as section, and they both stiffen the web plates and relieve the blank plainness of the exterior surface breaking it up with graceful shadow lines.

The lateral bracing along the extrados is designed for a wind pressure of 50 lbs. per sq. ft. on the arch ribs, and the flange is reinforced against bending at the portal by carrying the end strut angles down to the skewback. In addition to this the transverse sway bracing at every hanger is designed to be competent to carry to the floor panel one half of the wind load on the arch. The floor, being a monolith 57 ft. wide, constitutes a very rigid lateral beam to resist wind pressure. It has two expansion joints with steel aprons, one at each intersection of the floor with the intrados of the arch. The floor beam at this point is fixed to the arch and the floor stringers rest upon and slide on seat brackets to which they are bolted through slotted holes. As this joint is exposed to locomotive gases it was desirable to encase it as completely as possible in concrete, and this was accomplished by the use of a coating of clay 1 in. thick over the floor beam casing when the stringer casing was cast, which was in hot weather, and the clay was in large part picked out afterwards. A board form would have been very difficult to remove from the narrow space.

The light longitudinal struts connecting the floor hangers to reduce vibration are bolted to the arch through slotted holes.

The railing or parapet is a composite construction of zinc plate and concrete filling. The panels are solid webs of stamped sheet zinc ⅛-in. thick. The hand rail and the base rail are each formed of ⅛-in. sheet zinc and filled with concrete. The hand

rail is circular 4 ins. in diameter and the base is triangular in shape 3 ins. wide. The concrete core was intended to give stiffness and prevent denting by accidental or other blows. Both objects have been attained.

The ornamental finials of the posts are zinc castings, and the posts themselves are heavily galvanized steel angles. The railing is rigid and ornamental and protects the bridge deck from the clouds of railroad smoke and steam, and is no more costly than the closed cast iron railings heretofore used in such circumstances, and it is believed to be durable without requiring painting.

The total cost of the bridge was $78,500, which included considerable street and sidewalk paving on the approaches. The cost of the bridge proper per sq. ft. of floor was $4.30.

With the floor exposing only concrete to the action of the locomotive gases, and the railing presenting a surface that is only slightly corrodible, and all exposed steel work easy of access for painting, the structure should entail very slight expense for permanent maintenance.

CONSTRUCTION PROBLEMS IN REINFORCED CONCRETE BRIDGES.

By Walter M. Denman.*

In building quite a number of reinforced concrete bridges I have encountered some construction problems which may be of interest. In New England, perhaps more than in any other part of the country, we find difficulties in the reconstruction of bridges such as are seldom met with in the newer and less settled sections. Often both sides of the stream are occupied by buildings and we must accept conditions as they are and not as they ought to be. It often seems as though the only spot the early roadmakers could find for crossing a stream was just above the only suitable location for a mill-dam and as it is extremely difficult to change road layouts at this later day, the expense of foundation work is greatly increased. I shall endeavor to illustrate some of these obstacles to economical work as well as some construction problems and the manner in which they were overcome.

The bridge shown in Fig. 1 is the electric railway bridge under construction over the Deerfield River at Shelburne Falls, Mass., and is interesting for several reasons. First, because of the low cost of construction, $18,175. The bridge has 5 spans of from 70 to 84 ft.; its capacity is a train of 75 ton cars and the lowest competitive bid on a steel bridge of the same capacity on third-class masonry was $18,000.

The illustration shows a method of construction which has been found to be the only one which will avoid cracks in the spandrel walls. The usual practice is to build the arch-ring for its entire width and then erect the spandrel walls on top of it. Here the arch-ring may be seen built inside the spandrel walls with the forms for the latter ready to be poured on the fourth span and the walls completed on the first three. A careful examination will show that the section immediately above the piers still remains unbuilt so that expansion joints may be formed.

*Consulting Engineer, Springfield, Mass.

If the spandrel is built on top of the arch-ring as is the common practice, nine times out of ten the joint will open after centers are struck, and, in addition to the crack, moisture will seep through and stain the face of the ring. On the other hand, if the end of the arch-ring is built monolithic with the spandrel, a slight compression of the footings has no effect whatever upon the appearance of the finished bridge.

In Fig. 2 a good idea is obtained of the skew of the Shelburne Falls bridge although it cannot be observed that each span is on

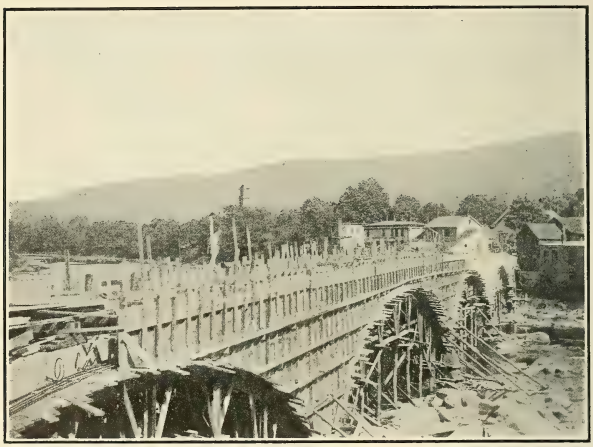

FIG. 1.—REINFORCED CONCRETE ELECTRIC RAILWAY BRIDGE, SHELBURNE FALLS, MASS.

an entirely different skew. This view is taken from the watch tower of the local fire department and affords an unusual picture. The angle of crossing at the river bank on the east side is 34 degs. and on the west side 48 degs. Not only does the river current change from one side to the other, but buildings and retaining walls on either side made it necessary to fit the bridge to it. The variation in skews was equally divided between the 5 spans or more properly between the 4 piers, which were wider up-stream than at the lower end. This bridge was built complete in 85 days.

Another bridge which offered some problems was that erected over the Kennebec River at Waterville, Maine, for the Lewiston, Augusta and Waterville Street Railway. This bridge is parallel to and immediately down stream from the old steel highway bridge joining Waterville to the town of Winslow. In the first design no consideration was taken of the steel bridge; but the railway officials demanded that the piers of the new bridge be placed adjoining the

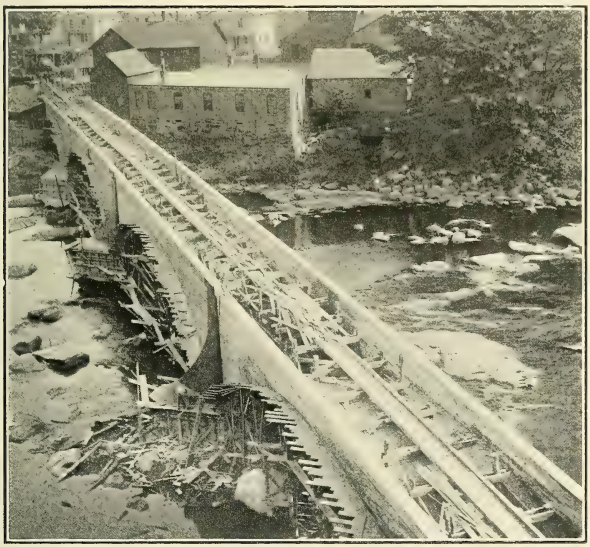

FIG. 2.—REINFORCED CONCRETE BRIDGE AT SHELBURNE FALLS, MASS.

steel bridge piers. This did not work for the most economical or sightly structure and to produce it safely and at a reasonable cost, a unique feature was introduced.

It will be noted that the second span from the left end is much shorter than any of the others. It is 85 ft. in length while the other three range from 130 to 135 ft. in length. In order to balance the thrust of the two long arches adjoining, the crown of the short span was dropped 8 ft. below the other arches and loaded

with fill. The result justified the means and the design has proved to be correct.

In Fig. 3 is seen the result of that part of a contractor's risk which can never be included in the estimate—the flood. The rise of the water in this case was most unexpected both as to season and height. It was outside the experience of the oldest inhabitant that the river should be high in September and never had such a flood been known except during the spring freshets. In this case the centers would have readily withstood the water, but they could not

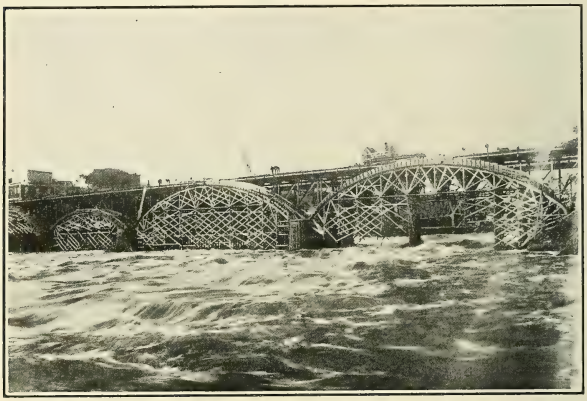

FIG. 3.—EFFECT OF FLOOD ON FORMS FOR REINFORCED CONCRETE BRIDGE AT WATERVILLE, MAINE.

resist the thousands of saw-logs which came down when the up-river booms broke. The posts in the centering were 8 x 8 ins. of sawn timbers and 6 bents of 8 posts each were sheared off like so many matches. In no case were whole sticks carried away at the joints, but each timber was broken.

The contractor had been ready to run the arch-ring on this span the day the flood came. The reinforcement was placed and the bulk-heads up. Fig. 4 shows the span the day after the flood and it will be observed that it is several feet out of line. The old highway bridge proved a friend in need at this time for lines were

made fast to it and the centering span pulled back into line; new posts were put under it and no very severe loss experienced.

The highway bridge was very convenient also for delivering material to the new bridge as shown in Fig. 5. A standard gauge track was laid from the mixer for the full length of the steel bridge and a hoisting engine and derrick mounted on a flat car. Two

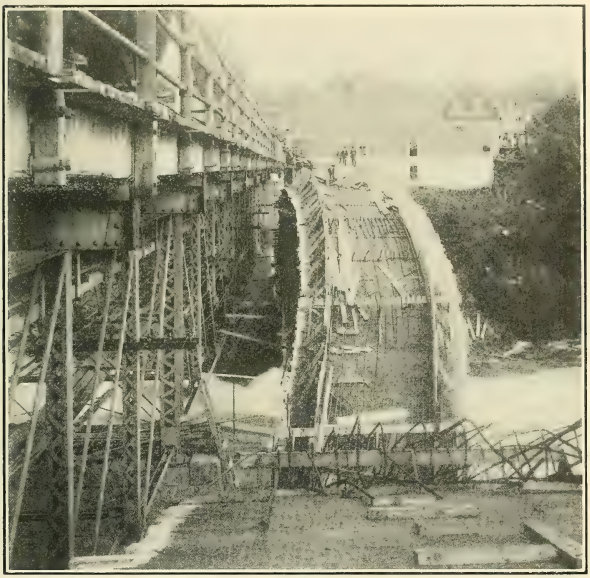

FIG. 4.—FORMS THROWN OUT OF LINE BY FLOOD. BRIDGE AT WATERVILLE MAINE.

24-in. gauge tracks were laid and the concrete delivered in one yard steel cars. All the form lumber was swung in with the derrick and part of the concrete placed by drop-bottom buckets while the balance was placed through wooden spouts.

A good idea of the height of the bridge is given by Fig. 6. It is 56 ft. from the water to the roadway and but 18 ft. from water level to top of piers. It may therefore be appreciated that

it was some task to build the spandrel walls with a length of 135 ft. and a height of up to 38 ft. on wooden forms and at the same time keep them straight. It was done, however, and not only are they

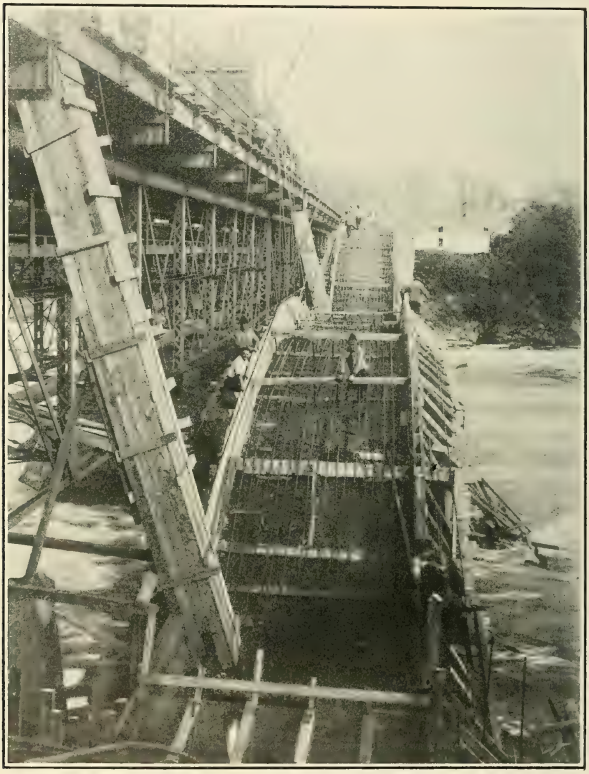

FIG. 5.—CENTERS AND STEEL READY FOR CONCRETE. BRIDGE AT WATERVILLE, MAINE.

straight but they are almost without a crack, one fine hair-line near the left end being the only such defect in the whole bridge after two years. It might also be noted that the temperature at

Waterville varies from 30 degs. below zero to 90 degs. above. So I think we are safe in saying that no ordinary temperature change has any effect on a properly designed concrete bridge.

Fig. 7 shows the plant used for building the Waterville bridge. Gravel was used exclusively in the concrete and was brought by train from the pit some 9 miles away. It was dumped into a hopper and elevated to the bin by a bucket elevator. The bin held 150 cu. yds. or about a 12 hour run. Two mixers received the gravel by gravity, the quantity for each batch being measured in a hopper just above the mixer. The concrete was then dumped into the cars or drop-bottom buckets and pushed out on the bridge.

FIG. 6.—CENTERING AND FORMS FOR REINFORCED CONCRETE BRIDGE AT WATERVILLE, MAINE.

An attempt was made to run this plant by electricity; but the loads were so variable that blowouts were of hourly occurrence and steam power was installed.

I have said that so often we have had to rebuild a bridge which crossed the stream just above a mill-dam and this case is exemplified in the bridge constructed for the Maine State Highway Commission at Sebec, Maine. Sebec Lake is some 12 miles long and is drained by the Sebec River. This bridge was built just below the narrows where the lake empties into the river. A timber and gravel dam was thrown across the narrows about 300 yds. above the bridge and for 3 weeks practically the whole lake was held;

so that by opening the gates of the dam below the bridge little water interfered with the foundation work.

The problem in this case was the use of the old abutments to resist the thrust of the new arches. We knew where ledge was to be found and as the old abutments were built of large blocks of granite which were assumed to be on ledge, it was decided to use only enough concrete in front of them to carry the vertical load and depend upon the weight of the abutments and the settled fill behind them to withstand the thrust.

Fig. 8 shows this bridge the day the centers were struck when

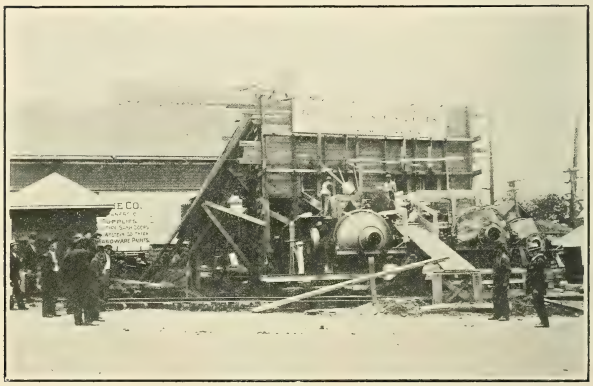

FIG. 7.—CONTRACTOR'S PLANT. REINFORCED CONCRETE BRIDGE AT WATERVILLE, MAINE.

I watched very carefully for an indication of movement but there was none of any kind. Of this I am quite positive as the rail had been built before the centers were struck and there was no sign of a crack. This is not good construction practice as a rule, but high water had been experienced and the only place the contractor could work was on the rail. Too often the rail is built before the fill is on and the centers struck and a badly cracked rail is the usual result.

One of the most interesting bridges I have had was that built over the Merrimac River at Nashua, N. H., in that it exercised

all the ingenuity of both engineer and contractor to put it through successfully. This bridge is the only one crossing the river between Tyngsboro and Manchester, a distance of 16 miles, and a large amount of traffic goes over it. In addition to the pedestrians and vehicles, three lines of electric cars converge at the east end and it was necessary to provide means for taking care of this traffic.

During the fall of 1909 plans had been prepared for a new bridge of steel and for the rebuilding of one pier and one abutment and bids had been received by a committee composed of five men from Nashua and five from Hudson, the town on the east side of the river. The low bid on these plans was $66,500 and the

FIG. 8.—REINFORCED CONCRETE BRIDGE AT SEBEC, MAINE.

committee were disposed to award the contract. The voters of Hudson objected, however, and elected a Board of Selectmen and a new committee with instructions to get the cost of a concrete bridge and report to an adjourned Town Meeting.

I was called in and looked over the site. The old bridge was what is known as a parabolic truss bridge in three spans of 150, 168 and 176 ft. and was built in 1882. It was in very fair condition as to the upper part of the trusses but, as is usual with highway bridges, the floor beams and stringers were in a dangerous condition, having been constantly neglected. I decided that it could be used for maintenance of traffic and gave the committee a guaranteed estimate of $80,000 for the total cost of a concrete

bridge. Plans and specifications were prepared, bids received and the contract awarded to Fred T. Ley and Company, Inc., of Springfield, Mass., for $74,485.

The first illustration of this bridge (Fig. 9) was taken about 6 weeks after active work was undertaken. It will be seen that the old bridge has been blocked up on the two middle piers; the piling for centers and foundations have been driven and the forms for two arches are pretty well along. The old bridge had to be raised 7 ft. on the piers and 5 ft. at the abutments. The raising was done by means of 50-ton Norton jacks applied at the end posts. Holes were cut in the cover-plates of the posts and 20-in. I Beams inserted. The post was then filled with hard pine timbers carrying the strain up to the top of the post and the jacks set under

FIG. 9.—CENTERING AND FORMS UNDER CONSTRUCTION FOR REINFORCED CONCRETE BRIDGE AT NASHUA, N. H. THE OLD STEEL BRIDGE HAS BEEN JACKED UP 7 FT. AT PIERS AND 5 FT. AT THE ABUTMENTS.

the I Beams. Two posts on one end of a pier would be raised about 4 ins. and then the jacks moved to the other side which would be raised as much. Even this small amount caused some trouble and danger.

The portal struts of the old bridge were far too light in the first place and the alternate raising of each side soon caused them to buckle while the end posts showed a tendency to work out of plumb. As all the stresses from the trusses were carried to the tops of the end posts, it will readily be seen that this movement out of line was very hazardous. Particularly was this the case with the long Nashua shore span where the old abutment had to be entirely removed.

In order to take care of this span, piles were driven in front of the abutment and back of it through the highway and the end posts were transferred onto new plate girders about 30 ft. in length. As soon as the transfer had been made, the old abutment was removed and piling driven to receive the new abutment. Naturally this made the structure less solid and increased the already great vibration.

In Fig. 10 is shown the supporting girders and also the method

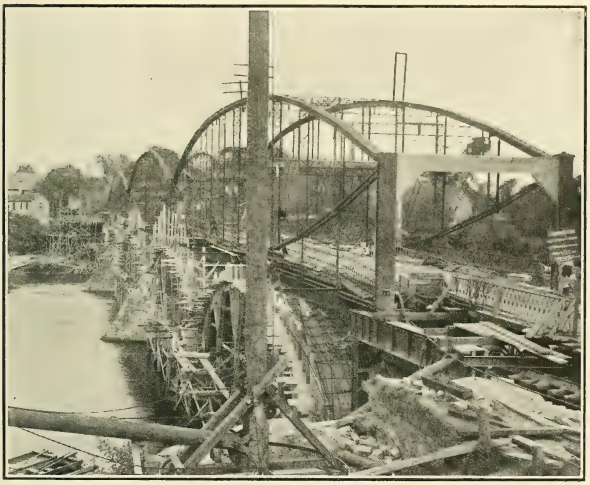

FIG. 10.—PORTAL STRUTS ENCASED IN CONCRETE. OLD STEEL BRIDGE AT NASHUA, N. H.

used to strengthen the portal struts. These latter were boxed and concreted in place and this end portal was stripped in about 10 days to note the behavior of the concrete. It is often specified that concrete must not be walked on or in any way shaken for at least 24 hours after being placed; but here was a case where the whole bridge was in a continual state of vibration for at least 18 out of the 24 hours and this concrete set hard and quickly and there was no crack to be found in any of these struts. It even

adhered to the end posts where the knee-brace came down, so firmly that it had to be broken away with heavy mauls when the old bridge was wrecked after completion of the new. It also acted as hoped for, in that it strengthened the portals enormously and reduced the vibration at least 50 per cent.

While the old bridge was out of the way between the old piers, we still had to contend with the end posts and their blocking when ready to run the arch-rings which passed over the old piers. This

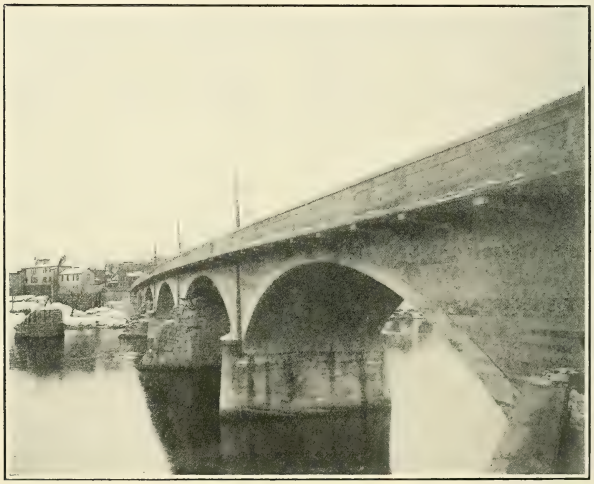

FIG. 11.—REINFORCED CONCRETE BRIDGE AT NASHUA, N. H., WITH FORMS REMOVED AND SHOWING CANTILEVER SIDEWALK.

was accomplished by sawing off the ends of the blocking, leaving a wood column of about 4 x 5 ft. which was boxed somewhat in hopper shape. The reinforcing bars were turned up at these boxes and the concrete run about them. Upon completion of the spandrel walls, the new bridge was completely filled with the exception of the space within these boxes,—the old iron bridge let down on the fill, and the trusses removed. After the trusses and end posts were taken out there still remained the old floor system and the

passage of teams was stopped for 7 days while this floor was taken out, grading finished and the car track laid. Foot passengers continued to go back and forth as they could use the sidewalk of the new bridge while the floor was being removed.

After the fill was completed, 3 of the holes in the arch-rings were filled with concrete over each old pier but the fourth hole was left open for a time. An elevator was built through it, a hoisting engine set upon the new fill and the stones of the old pier were lifted through this hole, loaded on wagons and hauled away. The hole so left is shown in Fig. 11, which view shows the bridge completed with the exception of these two holes and the surface finish. It also shows the cantilever sidewalk.

The bridge has five spans of 88, 96, 100, 96 and 88 ft. The driveway is 30 ft. clear and the bridge has one sidewalk with a width of 6 ft. The capacity is 100 lbs. per sq. ft. on the walk and 150 lbs. per sq. ft. on the driveway and under the track that of a 50-ton electric car. All wires, gas and water mains are laid under the roadway. The surface treatment, which had to be delayed until the spring of 1911, will consist of alternating rubbing and tooling the arch-ring, spandrel, coping and panels.

While these by no means represent all the difficulties that have been presented in the construction of somewhat over one hundred arches they are, perhaps, the more unusual and interesting.

A TEST OF A FLAT SLAB FLOOR IN A REINFORCED CONCRETE BUILDING.

By Arthur R. Lord.*

The most important recent development in reinforced concrete building construction is the introduction and rapid extension of the flat slab type of floor. The advantages of this construction, from the standpoint of both the occupant and the builder, are very generally admitted. It cannot, however, hope to receive the full confidence of engineers until the nature of the structural action in such floors is understood. Several attempts at analysis have been made but none have met with general acceptance. Actual designs in this construction offer some of the most striking contrasts and disagreements to be found in engineering practice to-day. It would seem that, by common consent, this construction, along with the voussoir arch and the retaining wall, must be classed as subjects beyond the range of pure analysis. In the presence of so many unknown conditions as obtain in the flat slab floor no analysis can stand which is not supported by convincing test data.

The question arises as to the means by which such tests may be secured most effectively. It is evident that a really satisfactory test of such construction can be carried on in our present laboratories only at a very large expense and at a sacrifice of many of the conditions obtaining in building construction. It may certainly be said that such a test as is practical under laboratory limitations would fail to carry the weight among practicing engineers that would attach to a test of an actual structure, in which laboratory methods and refinement prevailed. When a laboratory is taken into the field, however, there must be recognized the disadvantage arising from the fact that the test may not be carried to destruction. In such a test information on working stresses only can be secured, and it may not be possible to determine abso-

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lutely the weakest points in the design. However, there can be obtained much data of great value in a hitherto unexplored field, and one of the most important features of the test described in this paper lies in the fact that it demonstrates the feasibility of making field tests with practically laboratory refinement. A test which shows that it is possible to take precise extensometers into a building and measure the actual deformations which occur in the component parts of the structure as the load is applied or removed, has justified its undertaking. When the simple transformation of these deformations into stresses has been made, there is at hand most valuable information as a basis of future design. Questions regarding the fixedity of the slab at the column capitals, questions as to the extent of "arch" and "slab" action, and the other disturbing elements which separate analysis assumptions from actual conditions, are all overcome in the test of the actual structure and the true stresses present in the floor are observed directly.

In this paper such a test for actual stresses in a reinforced concrete building is described. The full possibilities of the method have not, of course, been realized in this its first application to building construction. An attempt was made to cover a few of the most essential points thoroughly, leaving many other questions for consideration in future tests when the experience gained in this test may be available for use in determining the best methods. Measurements were taken of the deflection at several points, of the deformation of the reinforcement at the center of the span, and over the column capital, of the deformation of the concrete at the edge of the capital, and of the curvature of the elastic surface in two directions. Working without precedents, it was necessary to design new instruments for use on this test and new methods of attachment for other instruments, so that it must be expected that the accuracy of the results was not so great as may be obtained under the best laboratory conditions. As will be shown, however, the stresses were determined within quite narrow limits and the difficulties were found less troublesome than had been expected.

Test Structure.—The test was made possible through the united action of the Leonard Construction Company, the Concrete Steel Products Company, both of Chicago, Ill., and the Engineer-

ing Experiment Station of the University of Illinois. The building tested was an eleven story and basement warehouse under construction at Minneapolis, Minn., for the Deere and Webber Company of that city. Fig. 1* is a photograph of the building at the time of test. Fig. 2 shows the floor plan of the building and

FIG. 1.—BUILDING FOR DEERE AND WEBBER COMPANY AT THE TIME OF TEST.

the location of the panels loaded. The dimensions of the panels are 18 ft. 8 ins. by 19 ft. 1 in. A 1:2:4 mixture was used, the slab thickness measuring 9 3/16 ins. The floor was designed for a live load of 225 lbs. per sq. ft., and the details of the reinforcement, shown in Fig. 3, are typical of the standard practice of the Concrete Steel Products Company. The floor tested was the

*Acknowledgment is made to *Cement* for the cuts of Figs. 2, 3, 9 and 10-14, used in this paper.—ED.

fourth from the ground and the conditions were not such as to make a high showing of strength. Owing to a failure in the supply of aggregates during the construction of this floor an abnormal number of bulkhead separations occur in the slab, as is shown in Fig. 2. Such separations occur in every panel under load except one. The concrete was only 40 days old at the begin-

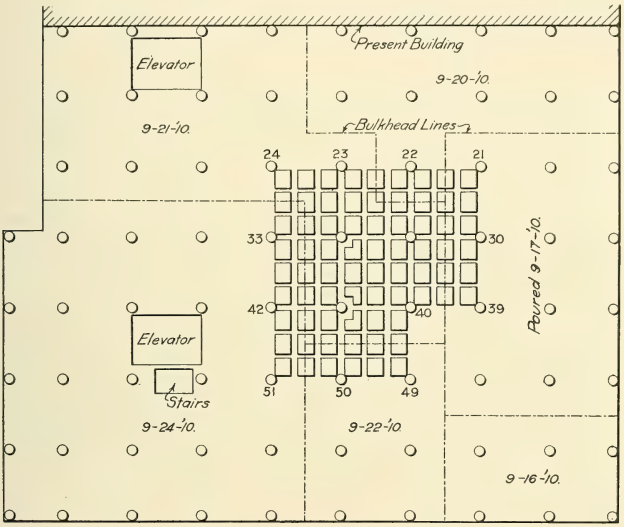

FIG. 2.—PLAN OF FLOOR SHOWING LOCATION OF PANELS TESTED.

ning of the test. In general the conditions were such as to give slightly higher stresses than would be expected were the slabs well seasoned and normally poured.

Method of Loading.—In applying the load care was taken that no serious arch action in the load be possible. In the earlier stages bricks were piled in piers, as shown in Fig. 2, and in Figs. 4 and 7, with open aisles from 8 to 16 ins. wide between piers. Later, cement was used as load, the piers being maintained separate as before. The load given in the tables is in all cases the

total load on the panel divided by the area of the panel, the *intensity* of load under the pier area being greater.

Instruments.—A brief description of the instruments for measuring deformations may be of assistance in understanding the methods used and in judging the accuracy of the results.

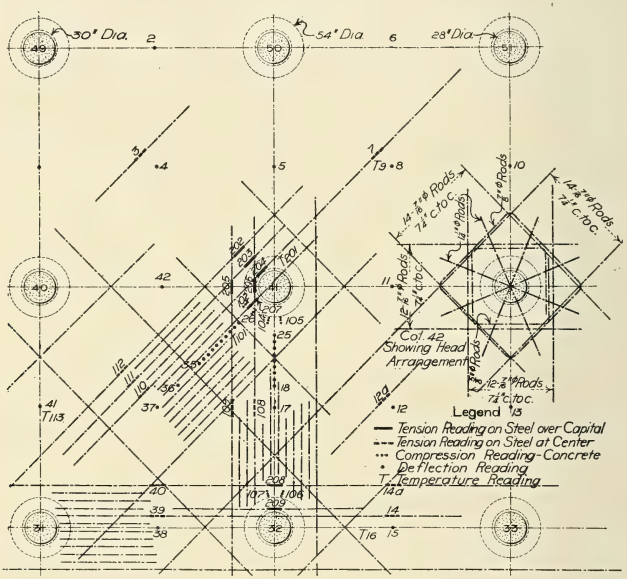

FIG. 3.—ARRANGEMENT OF REINFORCEMENT AND LOCATION OF OBSERVATION POINTS.

For deflection the instrument shown in Fig. 5 was used. A polished steel ball was attached to the ceiling and another carried on an upright, and the instrument was inserted between them. Measurements were made in this manner to the nearest .001 in. with accuracy. For measuring the deformation in the reinforcement at the center of the span a clamp was rigidly attached to the slab rod (the concrete being removed at one

point for this purpose), and a Wissler dial was carried on the clamp, Fig. 6. A fine, silk-covered, copper wire was attached to the rod at a distance of exactly 15 ins. from the clamp and passed immediately below the rod, over an idler on the clamp, and then over the drum of the dial. This wire was 1/16 in. below the under surface of the slab rod, and the deformations observed were therefore slightly in excess of the actual deformation in the rod. The wire was placed in this position because experience in the laboratory has demonstrated that measurements taken below the slab depend-

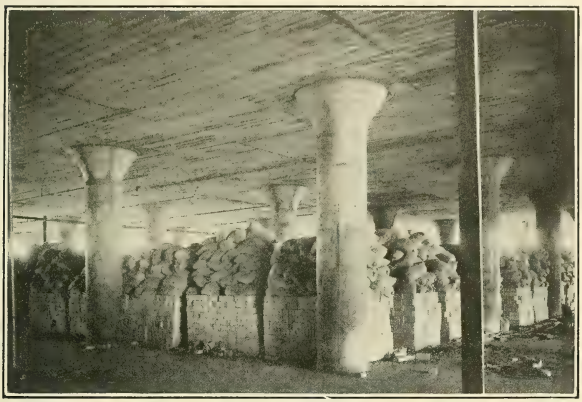

FIG. 4.—MANNER OF APPLICATION OF LOAD, SHOWING MAXIMUM LOAD APPLIED.

ing on the position of the neutral axis for correction, are subject to considerable error. By this arrangement the deformation was measured to an indicated reading of .0002 in. on a gauge length of 15 ins. The instrument was less responsive to slight changes than were the other instruments used, and the stress in the reinforcement may vary by as much as 800 to 1,000 lbs. per sq. in. from the value observed, which is much closer than can be designed for on account of the many unknown factors entering into all design computations.

For measuring the deformation in the reinforcement over the

column capital a modification of the Berry extensometer was made (with the permission of Professor H. C. Berry of the University of Pennsylvania). This instrument is shown in Fig. 7. The slab rods were first bared at two points 15 ins. apart and very small holes drilled in them. The points of the instrument fit into these holes and any deformation within the 15-in. gauge length is multiplied 5

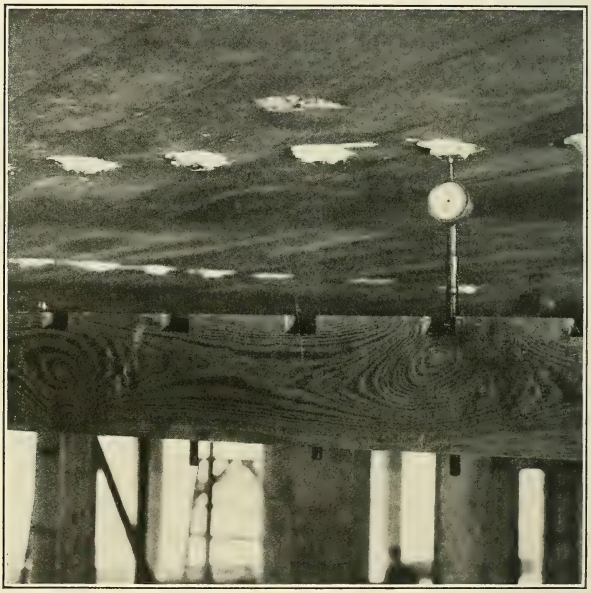

FIG. 5.—DEFLECTOMETER IN PLACE.

times by the lever arrangement before being read on the dial attached to the frame. A slight variation is possible in manipulating this instrument and 5 readings were therefore taken at each observation and averaged. The stress is obtained within a range of less than 1,000 lbs. per sq. in. A regular Berry 6-in. extensometer (Fig. 8) was used to measure deformations in the concrete

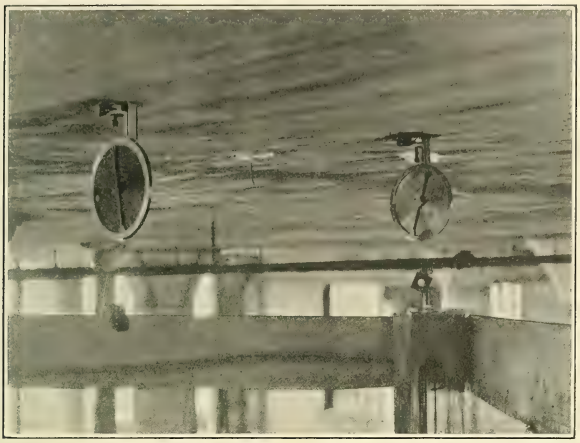

FIG. 6.—WISSLER DIAL FOR MEASURING DEFORMATION IN REINFORCEMENT.

FIG. 7.—MODIFIED BERRY EXTENSOMETER IN PLACE FOR MEASURING DEFORMATION IN REINFORCEMENT OVER CAPITAL.

over 6-in. gauge lengths. These observations are reliable certainly to the nearest 50 lbs. per sq. in. of stress.

Fig. 3 shows the distribution of points at which measurements were made over four of the panels. The numbers given are those used in recording and plotting the data in the diagrams and tables. The diagram is believed to be self-explanatory. The total number of readings was in excess of 3,300. The falsework for instruments and observers is shown in Fig. 9.

Effect of Temperature Changes.—To correct for temperature variations one entire day was spent in observing effects due to temperature alone, and the large Berry extensometer was read on a standard bar between each series of slab readings. Necessary

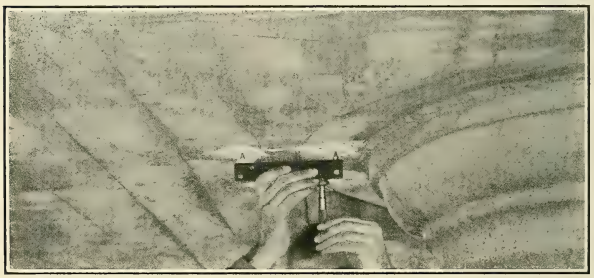

FIG. 8.—BERRY EXTENSOMETER.

corrections have been made in the tables accompanying this paper. In general temperature effects were much less serious than was anticipated.

In the planning of the tests and the designing of the instruments the writer was assisted by Professors Arthur N. Talbot and Herbert F. Moore of the Engineering Experiment Station of the University of Illinois. It should be noted also, in passing, that the success attending the actual taking of the observations was due in large measure to the fact that all the observers on the test were men of wide testing experience, and thoroughly familiar with the instruments handled. This condition is essential to the successful conduct of such a test. Observations were taken by Professors

H. F. Moore and W. A. Slater, and the writer, all of the station staff of investigators. Professor Talbot was present at the beginning of the test.

Application of Load.—The test continued for 6 days from October 30 to November 5 inclusive, 1910. Eight panels were loaded as shown in Figs. 2, 4 and 7. Readings were first taken on all instruments with the floor unloaded and then a load equal to 75 lbs. per sq. ft. was applied over the entire eight

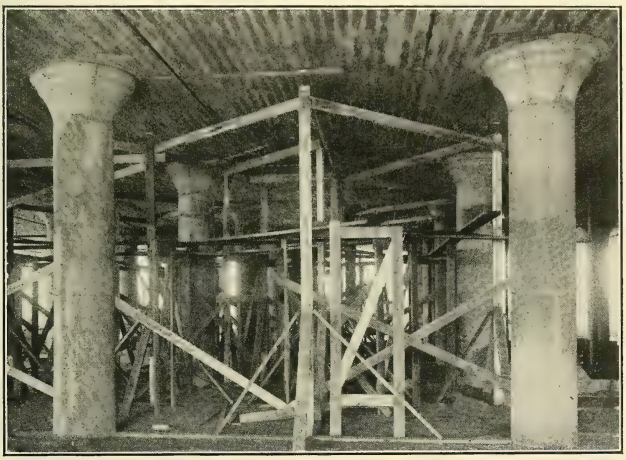

FIG. 9.—FALSE WORK FOR INSTRUMENTS AND OBSERVERS.

panels. Another set of readings was taken and the load increased to 150 lbs. per sq. ft. In this manner alternate readings and loadings were continued for 3 days. Over-night readings were taken in the evening, at midnight, and in the morning. The maximum load of 350 lbs. per sq. ft. was allowed to remain on the floor about 22 hours, readings being taken at frequent intervals during that time. The floor was then unloaded, the outer panels being first cleared, and finally the load was removed from the center panel. Readings were taken at intervals during the

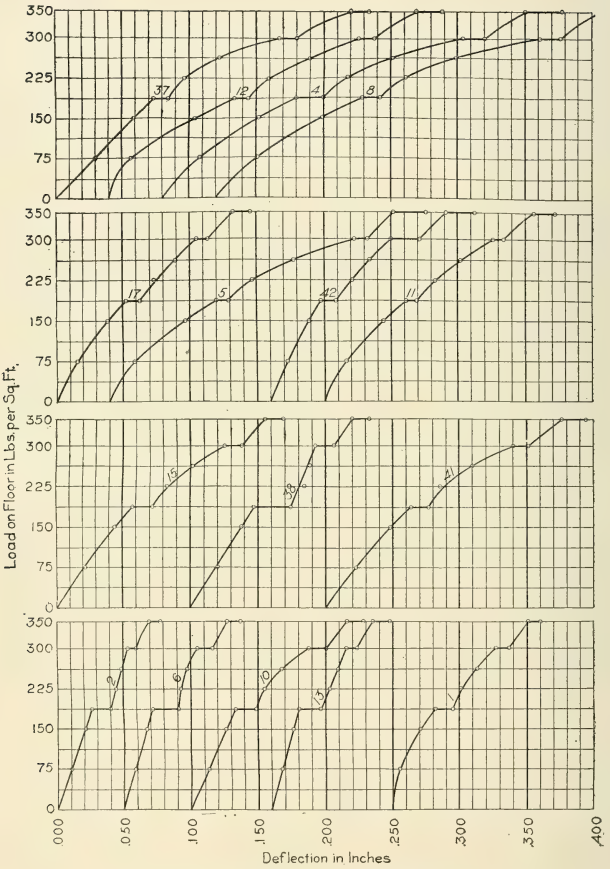

FIG. 10.—DIAGRAM OF DEFLECTIONS.

progress of the unloading. The data obtained are presented in Tables I-VI and plotted in Figs. 10-13. In the limited time since the completion of the test, and under the pressure of other experimentation, it has not been possible for the writer to give the data as thorough study as they merit. A few indications may, however, be pointed out from the diagrams.

Deflections.—Fig. 10 shows graphically the deflections at sixteen points, the same data being recorded in Table I. On the second diagram of Fig. 10 note the comparison between readings 5 and 11, where bulkheads existed, and readings 17 and 42, where no bulkheads were present. Other instances of the marked effect of bulkheads on the stiffness of the slab may be seen in the plotted data. It may also be said that in general the deflections were greater in the outer panels than in the center panels, due in part to the bulkheads in these outer panels, and in part to the natural tendency to higher stresses and deflections in end panels. The deflections would probably have been smaller with well cured concrete and in considering deflections it must be remembered that this slab was only 43 days old when the maximum load was placed upon it. The maximum deflection found was .32 in., exactly 1/1000 of the span. This was at a bulkhead in an outer panel. In the center panel the deflection, for all eight panels loaded, was .227 in., or 1/1400 of the span, which increased to .274 in. or 1/1200 of the span, when the load was removed from the outer panels.

Stress in Reinforcement at Center.—Fig. 11 and Table II give the data on the measured deformations in the reinforcement at the center of the spans. The table is reduced to unit deformations while the diagrams show total deformation over the lengths gauged. On the upper diagram in Fig. 11 are shown deformations in the center panel and it is to be noted that these are, in general, smaller than those in the outer panels. This would lead to the belief that the reinforcement at the center of the span should be designed for one panel loaded, as this apparently gives a worse condition *at the center* than full loading.

With reference to lower diagram of Fig. 11, it is of interest to point out that all three of these rods were not as high above the decking as the other rods in the band, and hence took a higher stress

TABLE I.—DEFLECTION OF SLAB (IN INCHES) AT POINTS MIDWAY BETWEEN COLUMNS.

Load in lbs. per sq. ft.		Observation Point No.															
Center Panel	Outer Panels	37	4	8	12	17	42	5	11	15	38	41	1	2	6	10	13
0	0	0.000	0.000	0.000	0.000	0.000	0.000	0 000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
75	75	29	29	32	16	16	13	18	18	22	20	24	7	11	11	15	10
150	150	61	73	79	66	39	30	57	45	44	38	49	22	22	17	27	17
187.5	187.5	74	101	111	95	52	35	81	63	58	57	64	33	26	19	34	20
187.5	187.5	0.085	0.121	0.123	0.105	0.063	0.049	0.089	0.070	0.072	0.076	0 077	0.046	0.041	0.041	0.050	0.038
225	225	98	139	143	119	74	61	107	82	83	83	86	52	45	43	56	45
262.5	262.5	124	173	180	149	89	75	138	103	103	93	111	64	48	45	69	50
300	300	168	225	242	186	105	101	183	128	126	93	140	78	48	57	88	57
300	300	177	234	252	193	110	109	189	132	135	99	148	80	52	62	96	62
300	300	0.180	0.242	0.258	0.198	0.113	0 112	0.193	0.134	0.139	0.106	0.152	0.087	0.059	0.068	0.101	0.064
350	350	224	272	293	231	133	132	218	157	156	121	178	97	70	77	117	76
350	350	217	281	301	235	135	131	224	161	161	123	182	102	71	79	121	81
350	350	220	285	306	240	137	138	227	163	162	125	186	101	73	79	122	80
350	350	223	288	309	242	138	143	229	163	162	125	186	102	73	80	121	80
350	350	224	.....	.....	244	139	137	231	165	163	126	188	102	73	81	123	81
350	350	227	.....	.....	246	141	140	235	167	165	128	191	103	74	82	125	83
350	350	0.234	0.299	0.321	0.250	0.146	0.146	0.238	0.169	0.170	0.134	0.196	0.107	0.078	0.088	0.130	0.090
350	187.5	246	223	241	188	138	134	172	124	125	127	181	80	63	70	104	73
350	0	266	115	114	97	124	112	72	50	54	115	156	34	58	41	58	43
350	0	274	116	111	96	126	117	68	47	54	122	160	36	42	46	61	48

than the band as a whole. The embedment of the various rods is given in Table V. The stresses observed at the center were very low and show how marked is the effect of slab action in decreasing

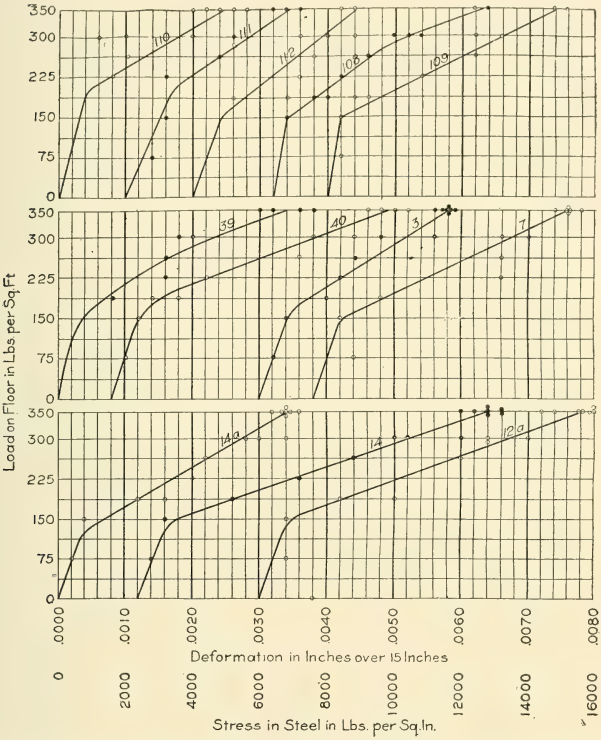

FIG. 11.—DIAGRAM SHOWING STRESS IN REINFORCEMENT AT CENTER OF SPAN.

the stress at these points. It would seem that any attempt to arrive at the values of these stresses without a liberal allowance being made for this slab action must result in misrepresenting the

TABLE II.—UNIT DEFORMATION IN REINFORCEMENT AT CENTER OF SPAN BETWEEN COLUMNS.

Load in lbs. per sq. ft.		Observation Point No.											
Center Panel	Outer Panels	39	40	108	109	14	14a	110	111	112	7	3	12a
0	0	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
75	75	− 1	1	− 3	1	0	1	− 5	3	− 3	− 1	1	− 3
150	150	− 1	3	1	1	1	3	− 5	4	− 4	− 3	3	− 3
187.5	187.5	0	4	4	3	8	8	− 3	− 1	1	1	7	3
187.5	187.5	0.00005	0.00007	0.00005	0.03004	0.00013	0.00011	−.00001	−.00001	0.00007	0.00007	0.00007	0.00008
225	225	11	9	7	9	15	13	5	4	8	12	8	1
262.5	262.5	11	19	9	15	20	15	7	9	9	12	9	15
300	300	21	20	12	15	24	19	7	3	11	12	12	17
300	300	13	20	12	13	25	20	4	3	8	15	12	17
300	300	0.00021	0.00024	0.00015	0.00017	0.00031	0.00023	0.00013	0.00011	0.00013	0.00021	0.00017	0.00021
350	350	21	27	20	24	31	24	15	17	13	21	19	23
350	350	20	29	20	24	32	23	15	15	12	20	19	24
350	350	21	25	21	24	33	23	13	16	13	20	19	28
350	350	24	28	20	23	35	23	16	16	12	20	19	27
350	350	25	27	21	23	33	23	19	16	11	19	17	28
350	350	21	27	20	24	33	23	17	12	9	19	19	28
350	350	0.00020	0.00025	0.00021	0.00024	0.00035	0.00023	0.00016	0.00011	0.00009	0.00020	0.00019	0.00027
350	187.5	19	25	23	26	31	17	19	9	12	17	17	19
350	0	17	28	21	26	11	− 1	21	15	15	9	4	3

actual conditions. The observed stresses indicate that the diagonal and cross band rods take practically the same stress.

Stress in Reinforcement over Head.—Table III and Fig. 12 give the data on this subject. The upper diagram covers diagonal rods, the lower diagram cross band rods. Note that rods 205 and 208, which are outer rods in the cross bands, took much higher

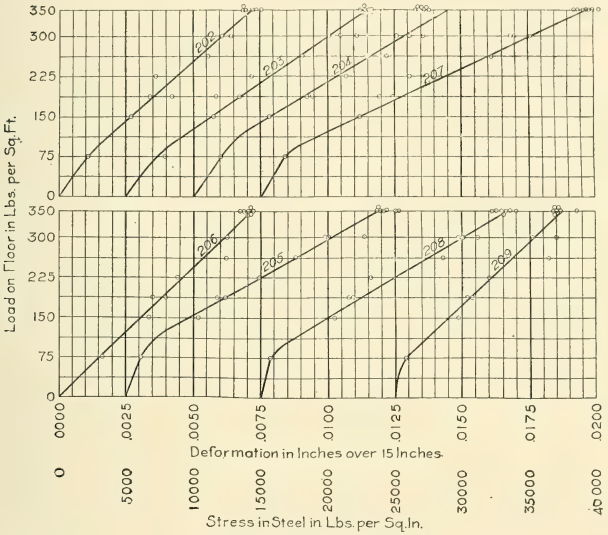

FIG. 12.—DIAGRAM SHOWING STRESS IN REINFORCEMENT OVER CAPITAL.

stresses than rods 206 and 209, which were near the center of the band. In the writer's opinion this is strong evidence against the assertion commonly made that these outer rods serve only to carry load to the rods which they intersect and which do pass over the column. Among the diagonal rods it may be noted that the stress in rod 207 was measured over the edge of the capital while that in 203 and 204 was measured opposite the center of the column. The higher stress in 207 would seem to indicate that the stress in these

rods decreases, passing from the critical section at the edge of the capital to any section nearer the center of the column. This is as would be expected. The stresses found from these readings indicate clearly that the slab should be designed for a maximum moment over the support, and not at the center as is quite commonly maintained.

In the design of this building some 75 per cent. more rein-

TABLE III.—UNIT DEFORMATION IN REINFORCEMENT OVER CAPITAL.

Load in lbs. per sq. ft.		Observation Point No.							
Center Panel	Outer Panels	202	203	204	207	205	206	208	209
0	0	.00000	.00000	.00000	.00000	.00000	.00000	.00000	.00000
75	75	14	8	7	6	4	11	2	2
150	150	24	22	19	25	18	22	19	14
187.5	187.5	29	28	30	33	24	26	23	18
187.5	187.5	00035	.00022	.00028	.00030	.00022	.00023	.00022	.00018
225	225	30	31	38	37	26	29	28	23
262.5	262.5	44	44	48	58	42	42	45	52
300	300	47	50	50	63	50	42	50	50
300	300	49	54	54	63	50	42	49	34
300	300	.00053	.00058	.00057	.00067	.00059	.00047	.00054	.00040
350	350	52	60	58	72	62	48	57	39
350	350	54	59	56	73	62	47	61	45
350	350	55	60	58	75	64	47	60	40
350	350	53	58	57	75	62	45	58	40
350	350	52	60	57	72	68	47	59	40
350	350	54	61	56	76	67	48	61	41
350	350	.00056	.00061	.00056	.00078	.00064	.00049	.00063	.00041
350	187.5	53	55	54	80	64	48	60	41
350	0	50	48	51	77	62	48	55	39

forcement was provided over the support than was used at the center, and it is easily imagined that the stress must be exceedingly high in many flat slab designs where this extra reinforcement is not provided in anywhere near the same amount. Since the stress reached the allowable value of 16,000 lbs. per sq. in., it may be concluded that the designers were not at fault in providing this additional reinforcement over the support.

It may be that a slightly greater amount of reinforcement

could be advantageously placed over the column head in order to reduce the stress at this point somewhat below 16,000 lbs. per sq. in., and make some additional allowance for irregularities in the actual execution of the design.

Stress in Concrete at Edge of Capital.—Fig. 13 and Table IV give deformations observed in the concrete. Owing to the fact that no intention of testing existed when the slab was poured no specimens of the concrete were available from which to determine the modulus of elasticity. Hence it is necessary to assume a value for concrete about 40 to 45 days old cured in fall weather at Minneapolis. From experiments made at the University of Illinois concrete of the same age cured under laboratory conditions showed a

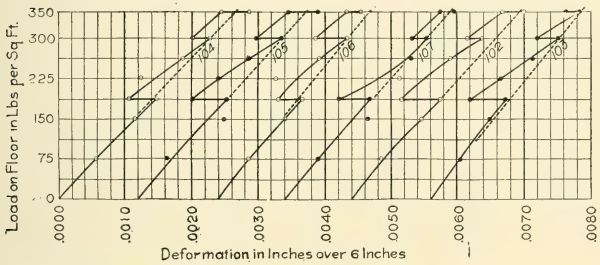

FIG. 13.—DIAGRAM SHOWING STRESS IN CONCRETE AT EDGE OF CAPITAL.

modulus of 1,875,000 lbs. per sq. in., and in Table VI this value has been used as giving the very highest possible value for the concrete stress. It would probably be fairer to the construction to use a modulus of 1,500,000 lbs. per sq. in., resulting in stresses 4/5 as large. In Fig. 13 a stress of 100 lbs. per sq. in. corresponds to a deformation of .0004 in. with a modulus of 1,875,000, or to a deformation of .0005 in. with a modulus of 1,500,000.

An interesting feature shown in the curves is the falling off in the concrete stress when the load was allowed to remain over night. The decrease is less marked at higher loads than at low loads, while readings taken at very frequent intervals while the maximum load was on the floor showed that at first the stress steadily

increased and the decrease did not begin until some time after the load was applied. The phenomenon is of interest as showing the readjustment in stresses which takes place under load even in the least plastic constructions. In general the concrete stresses checked those found in the reinforcement over the support. It would not seem that compressive reinforcement were needed at this point, the maximum stress being from 650 to 750 lbs.

TABLE IV.—UNIT DEFORMATION IN CONCRETE AT EDGE OF CAPITAL.

Load in lbs. per sq. ft.		Observation Point No.					
Center Panel	Outer Panels	102	103	104	105	106	107
0	0	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
75	75	11	17	9	7	8	7
150	150	21	28	21	22	17	20
187.5	187.5	24	30	24	16	18	20
187.5	187.5	0 00015	0.00021	0.00018	0.00013	0.00015	0.00012
225	225	15	25	20	17	14	20
262.5	262.5	28	37	31	28	25	31
300	300	36	43	38	36	32	34
300	300	39	44	38	33	30	33
300	300	0.00032	0.00038	0.00033	0.00029	0.00024	0.00031
350	350	41	47	40	38	32	38
350	350	43	47	42	39	33	38
350	350	43	49	43	42	35	39
350	350	44	45	44	42	34	37
350	350	47	48	46	42	34	40
350	350	49	50	48	41	36	40
350	350	0.00047	0.00045	0.00046	0.00040	0.00034	0.00040
350	187.5	47	49	44	36	31	37
350	0	44	44	38	23	22	30

per sq. in., which is permissible for concrete stressed in several directions and over a very short distance. If, however, a higher percentage* of reinforcement had been used, resulting in a thinner slab, as is commonly done, it is evident that compressive reinforcement would be absolutely necessary for good design.

* In the slab tested a very low percentage of reinforcement was used, giving a much thicker slab and hence better stress conditions at the support than is usually found in this type of construction. The writer is informed that this is the standard practice of the company which designed the building, but the need of this precaution does not seem to be at all generally recognized.

Summary of Stresses.—Table VI gives a summary of the stresses found at various points under the design load of 225 lbs. per sq. ft. and also under the maximum load applied of 350 lbs. per sq. ft. In every case where a choice has been possible the higher value has been given, as for instance in the concrete stresses the modulus has been assumed as 1,875,000 lbs. per sq. in. instead of the (probably) more just value of 1,500,000 lbs. per sq. in. The

TABLE V.—DATA ON POSITION OF RODS ON WHICH DEFORMATIONS WERE MEASURED.

Observation Point No.	Band.	Position in Band.	*Embedment, inches.	Layer of Steel over Column.
3	Diagonal	3d rod from center	$\frac{11}{16}$	
7	"	" " " "	$\frac{15}{16}$	....
12a	"	2d " " "	$\frac{3}{8}$	
14	Cross	" " " "	$\frac{5}{16}$	....
14a	"	Outer rod of band	$\frac{1}{2}$	
39	"	2d rod from center	$\frac{3}{8}$	....
40	"	Outer rod of band	$\frac{11}{16}$	
108	"	3d rod from center	$\frac{15}{16}$	....
109	"	Outer rod of band	$\frac{13}{16}$	
110	Diagonal	3d rod from center	$\frac{3}{4}$	....
111	"	5th rod from center	$\frac{13}{16}$	
112	"	Outer rod of band	$\frac{9}{16}$	....
202	"	" " " "	$2\frac{7}{4}$	2d layer from top.
203	"	5th rod from center	$2\frac{1}{16}$	.... " " " "
204	"	3d " " "	$1\frac{7}{8}$	" " " "
207	"	1st " " "	$2\frac{3}{8}$	.... " " " "
205	Cross	Outer rod of band	$3\frac{1}{4}$	3d " " "
206	"	3d rod from center	3	.... " " " "
208	"	Outer rod of band	$2\frac{3}{4}$	" " " "
209	"	3d rod from center	$2\frac{1}{4}$	.... " " " "

*Measurement from surface to center of rod.

dead load stresses have been taken as one-half the live load stress at the design load (the dead weight of the slab being half this load). This, also, is a maximum assumption and probably somewhat in excess of the true value, as the concrete was not broken in tension until after a live load of 75 lbs. per sq. ft. was applied, and the tension in the concrete will undoubtedly account for a much higher percentage of the dead weight than of the live load.

Cracks.—Very careful observations were made to discover and record all cracks. A powerful reading glass was used to aid

the eye, and dust was removed by means of bellows. It is exceedingly easy *not* to discover cracks in such a test, and an ordinary observer would very likely have been aware of the existence of but few of these cracks. At a load of 262.5 lbs. per sq. ft. a crack was observed at the bulkhead where two days had elasped between the pouring of the adjacent floor sections.

TABLE VI.—SUMMARY OF STRESSES.

		Design Load—225 lbs. per sq. ft.			Maximum Load—350 lbs. per sq. ft.		
		L. L.	D. L.	Total.	L. L.	D. L.	Total.
REINFORCEMENT OVER HEAD.							
Diagonal Band.	Maximum	13,800	6,900	20,700	24,200	6,900	31,100
	Average..	11,000	5,500	16,500	18,800	5,500	24,300
Cross Band.	Maximum	10,000	5,000	15,000	18,800	5,000	23,800
	Average..	9,000	4,500	13,500	17,200	4,500	21,700
REINFORCEMENT AT CENTER:							
Diagonal Band.	Maximum	2,400	1,200	3,600	4,800	1,200	6,000
	Average..	2,000	1,000	3,000	4,800	1,000	5,800
Cross Band.	Maximum	2,800	1,400	4,200	8,000	1,400	9,400
	Average..	2,500	1,300	3,800	6,600	1,300	7,900
Outer Panels.	Maximum	4,600	2,300	6,900	10,400	2,300	12,700
	Average..	3,800	1,900	5,700	8,000	1,900	9,900
CONCRETE AT CAPITAL:							
Diagonal Direction.	Maximum	530	265	795	800	265	1,065
	Average..	500	250	750	750	250	1,000
Cross Direction.	Maximum	500	250	750	800	250	1,050
	Average..	468	234	700	750	234	984

Concrete Stresses based on $E_c = 1,875,000$ lbs. per sq. in.

At 300 lbs. per sq. ft. cracks appeared at the other bulkheads. Suggestions of cracks were also found in the center panel where no bulkhead existed and over the edge of the capital at Column No. 41, these being very faint and hard to trace for any distance. At 350 lbs. per sq. ft. there could be traced out the cracks shown in Fig. 14, the dotted lines representing cracks in the ceiling below. The cracks about the column head are of interest as indicating the

position of the critical section for which moments should be figured in analyses. These averaged about 2 or 3 ins. outside the edge of the capital below.

At Column No. 51 the position of the crack would seem to indicate that for a single panel loaded the critical section moves

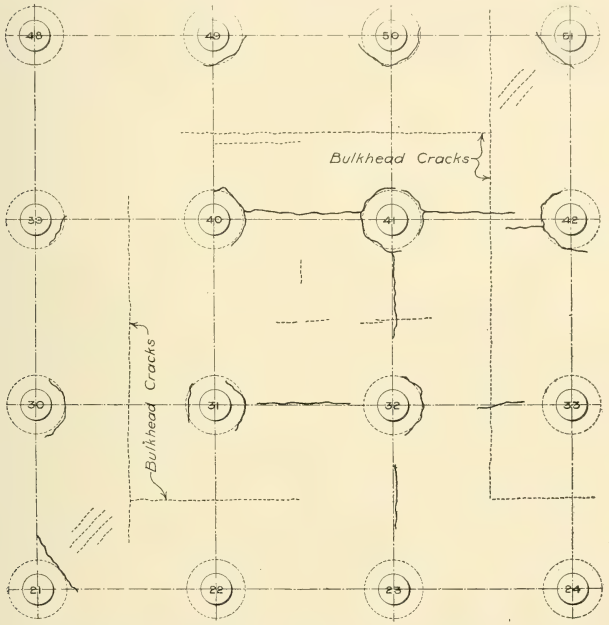

Fig. 14.—Location of cracks traceable at load of 350 lbs. per sq. ft.

nearer the support resulting in higher stresses at the center. This crack and similar ones, as at Columns Nos. 49, 39, and 21, were very faint, indicating a lower stress in the reinforcement over the support at such points. The cracks shown running diagonally near Columns Nos. 21 and 51 were in all cases directly beneath slab rods.

Another set of cracks which developed only under the maximum load of 350 lbs. per sq. ft. possesses a peculiar significance. These cracks ran along the center of the cross bands, being easily traced at the center, growing fainter toward the columns, and disappearing entirely in most cases before reaching the crack over the edge of the capital. These cracks afford a most satisfactory evidence of the presence of "slab action", and would be expected to occur if Professor Talbot's theory of such action is accepted. These cracks have, we believe, never been observed before, probably because other building tests have not been extensive enough to develop them to a marked degree, and because cracks have not ordinarily been very carefully observed.

General Conclusions.—One test does not permit of very conclusive deductions—it is hoped and believed that many such tests will be made in the near future, covering a sufficient range of panel sizes and live loads to afford conclusive design data. The most important result of the test, in the writer's opinion, lies in the demonstration that well-equipped laboratories may undertake such tests with the reasonable expectation of securing reliable and useful data. The difficulties to be met are not greater than those encountered in experimental work in the laboratory. It is not believed, however, that inexperienced men would be warranted in undertaking such a test as the instruments used require the handling of trained observers.

The test gives certain well-defined indications.

It indicates positively that moments are much greater at the support than at the center of the span.

It indicates, by the position of the cracks, the section for which moments should be calculated at the support.

It indicates that the stresses at the center of the span are much lower than the computed stresses at this point would lead us to expect.

It indicates that bulkheads act to increase deflections and stresses.

It indicates that for the critical section at the support, considering all of the reinforcement equally effective, the moment is much less than analyses would call for. Where the various analyses

give a moment of $WL/12$ to $WL/14$, the moment calculated from the stresses observed in the test is roughly only $WL/22$.

It indicates that the reinforcement *at the center* receives its maximum stress for the condition of load on one panel only.

The test shows that the design of this particular building is well balanced and that the stresses are safe. It does indicate, however, that serious questions may be entertained as to the stresses in many buildings of this type where higher percentages of reinforcement have been used and very little additional strength provided at the support.

A DISCUSSION OF THE BASIS OF DESIGN FOR REINFORCED CONCRETE FLOOR SLABS.

By Arthur R. Lord.*

In this paper the writer desires to state briefly, without attempting to make connections or applications, the evidence presented by the test of a flat slab floor in a reinforced concrete building†, and also such confirming evidence from other tests made recently at the University of Illinois, as is applicable to a consideration of the basis of design for reinforced concrete floor slabs. Owing to the limited time it will be necessary to simply state many conclusions. Points on which a division of opinion may be expected will be discussed more fully.

The division of the slab into beams or beam strips gives a method of analysis which appeals to many as the logical manner of approach in undertaking the analysis of the reinforced concrete floor slab. It does not seem, however, that the *true* stresses may be so arrived at, but we may safely conclude that the stresses so found will represent the *maximum* value.

It must be recognized that the stresses indicated by the beam method of analysis are reduced by slab action and by arch action to a very considerable degree, but it is not possible at present to state coefficients for this reduction or to give it quantitative expression. The amount of this reduction probably depends on the depth of the slab and on the details of the head and slab reinforcement, and may be expected to vary considerably under different conditions.

Certain lines of approach do not seem to be tenable as a basis of analysis. Whatever the value of Grashof's analysis of homogeneous plates fixed on a system of equidistant points may be (and this value is not clearly established), the use of the ratios from Grashof does not seem to be warranted without advancing weighty supplementary evidence. Other writers have pointed out the important bearing which Poisson's ratio has on the value of the

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†See paper on Test of a Flat Slab Floor in a Reinforced Concrete Building, by A. R. Lord, p. 156.—Ed.

stress found by Grashof's equations. The writer would mention also that the expression found by Grashof is indeterminate for other than square panels, and that the maximum stress is located at the center of the cross band (as applied to the floor slab) and not at the support, as Mr. MacMillan evidently supposes. This can hardly be said to represent the conditions in the reinforced concrete floor slab.

Treatments based on Dr. Eddy's analysis of the circular flat plate do not seem to apply for the same reasons. This analysis assumes certain relations in both shear and moment curves which, while tenable for the cases that he treats, do not seem to be applicable to a plate supported over a considerable area at its center. Dr. Eddy does not attempt to apply his analysis to any similar cases.

A study of stress conditions would indicate that certain limitations should be recognized in designing the flat slab. Beam analysis applied to the slab and the results of tests both indicate that the moment is greater over the support than at the center, and that the critical section is at or near the edge of the capital. If this be true, and we wish to adhere to the established practice of maintaining the same slab design for all floors, it is essential that the capital be also maintained at or above some minimum size, for which minimum size the moments have been figured. Common practice makes the capital size depend on the column size, when it should depend on the dimensions of the panel. The stresses in the reinforcement and concrete due to moment and the shearing and diagonal-tension stresses are all increased rapidly by a decrease in the size of the capital. Over the small capitals in the upper floors of buildings of this type the stresses must be much higher than they are for the lower floors with their large capitals. From a study of shear and moment stresses in several buildings it is recommended that the capital be maintained at a diameter of at least two-tenths the dimension of the side of the panel, if the capital is square, or at an equivalent area if the capital be of different shape. Shearing and diagonal-tension stresses will be safe for this size of capital provided a proper percentage of reinforcement is used in the slab design proper.

It is quite commonly held that only the rods which pass over

the capital may be taken into account in figuring the resistance to negative moments. Under this conception the strips containing the outer rods of a band serve only to carry load to other strips which they intersect and which do pass over the capital. In this conception it would seem that there is a confusion between transfer of weight and of stress. The weight or load follows the most direct path to the column, being transmitted in the form of shear in the concrete itself, and does not need to be transferred from strip to strip at their intersection. The wide beam while stressed very high at the support locally in shear, may be stressed over its entire width in moment. A series of tests made during the past year at the University of Illinois illustrates this lateral distribution of *stress* in wide beams. These beams were of various dimensions and proportions, and it may be permitted in advance of the full publication to present one particular set in which the proportions were very near to the conditions found in ordinary building design. The beams were 36 ins. wide, 4 ft. 10 ins. long, and 3 ins. deep to the reinforcement. Thus the ratio of depth to width is $^1/_{12}$ or somewhat in excess of the usual ratio of depth to reinforcement to width of band in the flat slab. The beams were loaded across their full width at the ends, and were supported at the third points, in some cases for their full width, in others for one-half of their width, and in still others for one-fifth of their width. The beams were made and tested in duplicate at approximately 60 days. The table gives the loads carried:

		Total Load Carried.		
Beam No.	Supported.	Beam No. 1.	Beam No. 2.	Average.
711.1 – 2	Full width	15,550	15,800	15,770
713.1 – 2	Half width	15,000	17,000	16,000
715.1 – 2	Fifth width	14,900	12,250	13,500
717.1 – 2	Fifth width	14,550	16,000	15,300

The 717 beams were provided with 0.4 per cent. cross reinforcement.

All the beams had 10 – ⅜ in. round rods 3 ins. from the face, and beams 717.1-2 had ¼ in. round rods 4 ins. on centers crosswise of the beam.

We may note that for one-half support there was no appreciable falling off in the load carried. The deflection curves showed also that the reinforcement at the edges took the same stress as did

the rods in the beam supported for its full width. The yield point of the reinforcement used was 37,000 to 38,000 lbs. per sq. in., and computations show that all the reinforcement present in both cases was stressed about 10 per cent. beyond the yield point when the yielding of the reinforcement caused the concrete to fail in compression. This is exactly the phenomena of the ordinary beam test where there is no question of lateral distribution involved. When the beam was supported for only one-fifth its width there was but a small falling off in the load carried.

For all cases where the recommendation of this paper with regard to column size is followed the beams or bands will be supported for one-half of their widths, in which case the entire reinforcement would seem to be effective.

The evidence of the wide beam tests is further confirmed by tests of footings. It may be pointed out that the portion of the slab inside the line of inflection is in effect an inverted footing loaded with a peripheral load at the line of inflection (reactions of the supported beams between the lines of inflection) and a uniform load over the base. The footings tested were of uniform depth throughout, being somewhat thicker in proportion than the cantilever slab about the column. From a consideration of all the results Professor Talbot has arrived at the conclusion that the width of the resisting section, as governing the stress in the reinforcement, was composed of the width of the pier, plus the depth to the reinforcement on each side of this, plus one-half of the remaining width of the footing. Applying this to several designs revealed the fact that this width includes all the reinforcement in both the cross and diagonal bands in all ordinary designs. In other words, not the reinforcement passing over the capital only, but the entire reinforcement in both bands may be counted on to take its full resisting moment. Actually observed stresses found in the building test bear out this statement. The stress was found to be nearly constant across the entire band, the slight falling off in the case of the outer diagonal rod being accounted for in part at least by its lower position in the slab. To offset this the outer rods of the cross bands were observed to be taking a slightly higher stress than those in the center of the bands.

The footing tests previously mentioned afford evidence also

as to the position of the section at which negative moments should be figured for any band. The reinforcement in the footings was laid sometimes in two bands and sometimes in four bands without any noticeable effect on the strength or the stresses found in the specimen. The tests have been interpreted for a bulletin on footings now in manuscript form and the conclusion was reached that stresses figured for a section at the face of the pier would represent fairly the structural action involved. Additional evidence was afforded from the observation of cracks about the columns in the building tested. These cracks formed at a distance on the average of about 2 ins. outside of the edge of the capital below, and would seem to indicate that moments may safely be figured about a section at the edge of the capital as giving a maximum condition.

If we consider the bands as continuous wide beams we should expect to find that the moment at the support was twice that at the center of the span for uniform loading. The test indicated that this center moment was somewhat less in proportion, but it is possible under a single panel loaded that the stress at the center may reach as high a value as one-half that at the support under full load, and this fraction may be considered a maximum for design purposes. Much current practice is in direct contradiction to this conclusion. This objection to any adaptation of Grashof's analysis has been noted in a previous paragraph. Mr. Turner's formula, based as it apparently is on tests of single panels and without the determination of actual stresses, must be applied with caution. When a formula is to be derived from tests it is essential that the tests be so planned to give as high stresses as may reasonably be expected under full load conditions. Certainly tests which from their nature predetermine low stresses at the critical section at the support do not constitute an acceptable basis for a design formula. Mr. Turner's contention that by the use of a uniform grade of open hearth steel it is possible to tell when the stress in the steel reaches the yield point, and that in this manner a basis of design may be arrived at, does not inspire one familiar with test phenomena with much confidence. It does not answer the objection to single panel loads at all. Again it is not clear that any noticeable change will occur in the slab when the steel at the support reaches the yield point. At this time the steel over the support starts to yield:

simultaneously the deflection and hence stress at the center increases, and thus relief is afforded the steel at the support. The final failure may even occur at the center, and yet it is evident that this readjustment should not be necessary in good design, even if there is sufficient excess reinforcement at the center to safely carry the added stress (which is not always the case). After the readjustment has taken place there are present in the reinforcement and concrete large permanent stresses and marked set, and against a repetition of the load the structure is even less secure than at first.

The test indicated that the line of inflection lay between a square having the column as its center and its inscribed circle. If the slab be considered as simply supported the least dimension of the line of inflection (in the cross band direction) would be approximately 0.44 L where L is the dimension of the panel. Owing to the fact that the slab is fixed for a considerable area at the column capital, the effective clear span is much reduced, and the least dimension of the line of inflection correspondingly increased. If the effective clear span is taken as the distance between the critical sections already determined for negative moment, which seems a reasonable basis, and if we follow the recommendation as to size of capital stated previously in this paper, the least dimension of the line of inflection becomes 0.55 L. [$0.2\ L + 2\ (.22 \times .8\ L) = 0.55\ L$; the $.8\ L$ being the effective clear span.] In the building tested at Minneapolis the capital size was 0.24 L and the observed least dimension of the line of inflection (in other words the dimension along the cross band) was about 0.56 L. On the basis given above it would figure at $0.24\ L + 2\ (.22 \times .76\ L) = 0.57\ L$, which checks closely with the observed fact. It would seem that for the conditions found in practice, and if columns are not permitted to have unduly small capitals, the variation in this dimension of the line of inflection would be small, and the value of 0.55 L here given may be quite generally applicable. The dimension of the line of inflection along the diagonal band was observed in the test to be about .64 L.

A question exists as to the value of the compressive stress in the concrete at the edge of the capital, and as to whether or not compressive reinforcement is needed. It is difficult to compute

the stress at this point owing to the fact that the concrete is stressed not in one, but in several directions. In the test, however, the resultant stress was measured directly, ranging from 650 to 750 lbs. per sq. in., which is undoubtedly safe for concrete stressed in several directions and over a very short distance. In the design of this building a very low percentage of reinforcement was used at the center of the span, and it would seem as though the use of this low percentage were imperative to proper design unless compressive reinforcement is to be used. It may well be asked what the stress in the concrete would have been had .5 per cent. or .6 per cent. reinforcement been used at the center, as is frequently, I might say commonly, done. For such practice compressive reinforcement seems to be required.

The writer would like to add a word on the question of shearing stresses in flat slabs. To the designer of beam construction these stresses have seemed high, and a proper consideration of shear, as well as of moment, makes it advisable to provide a larger capital than is ordinarily used on the upper floors of our present buildings. The recommended minimum size of capital of 0.2 L was selected with the shear computations of several buildings directly in mind. We should remember, however, that a sharp distinction should be made between so-called "punching" shear and shear which indicates the tendency to diagonal tension. Punching shear may properly be figured along the periphery of the capital giving a punching shear unit stress of

$$v = \frac{W\ (L^2 - c^2)}{12 \times 4\ cd} = 0.1\ \frac{wL}{d}$$

the last simple equation applying only when the capital is 0.2 L on a side as recommended (or a circle of equivalent area). In this formula

L = side of square panel in feet
c = side of equivalent square capital in feet
d = depth of slab in inches.

For such shearing stress the value recommended by the Joint Committee of 120 lbs. per sq. in. is certainly permissible.

In computing shear as indicating the tendency to diagonal tension, the flat slab should be given the same treatment which is

accorded to beam design. In a restrained beam the failure in diagonal tension does not occur at the support, but at some distance out from it. In like manner Professor Talbot has found in the tests of footings that the failure in diagonal tension took place some distance out from the pier, and he concludes that as an indication of the tendency to diagonal tension the *intensity* of shearing stress should be computed on a section at a distance out from the pier (or edge of capital) equal to the depth of the slab to the reinforcement. Using the same notation as before

$$v' \text{ (diagonal tension tendency)} = \frac{w\,[L^2 + (c + .167d)^2]}{12 \times 4\ (c + .167d)\ jd}$$

In any particular case it is simpler to figure this stress directly than to use the formula. For this value the various limits specified by the Joint Committee should govern.

It is a simple matter to state the above conclusions and recommendations in the form of an analysis, but it is not believed that analyses are profitable until some means is found for expressing quantitatively the amount of the reduction in stresses due to arch action and slab action. The amount of this reduction was observed in the test to be large.

DISCUSSION.

Mr. MacMillan.

MR. A. B. MACMILLAN.—I have been very much interested in the papers by Mr. Lord and the findings as to cracks, especially those around the heads of columns and between the columns, as in similar work I have observed practically the same thing.

In a number of cases cracks have appeared very close to the head of the column, particularly at the edge, as near as can be determined, and also between the columns in straight lines. Some of the cracks disappeared a very short distance from the column, but in no instance did they go from column to column. I think the cracks were caused in a large measure by the failure to place the reinforcement at the top of the slab. In ordinary practice the reinforcement is supported over the columns, as near the top of the slab as possible. The bars rest on the rings and radials placed in the column head for the purpose and are allowed to sag to near the bottom of the slab between the columns. This saves time in bending the bars but it does not make their correct position positive. I think this is one point which requires correction.

A test of a single slab was made by Mr. Knight C. Richmond, but I do not know how the slab was designed nor the amount of reinforcement. A brief summary of the test is as follows:

Test slab was 7 ins. thick, 20 ft. long, 17 ft. wide. The rectangular columns at corners were 15x12 ins. Load was applied with 90-lb. sand bags, laid to prevent arching. Concrete 1:2:4. The weight of the floor was about 85 lbs. per sq. ft.; and the total design load was taken at 150 lbs. per sq. ft.

Load lbs. per sq. ft.	Deflection ins.	Remarks
75	1/64	Test started June 20, 1910, at 11 A. M.
135	1/16	
225	3/32	
270	1/8	
360	5/32	Test was stopped at this time and the slab left loaded with the intention of continuing the test July 25, 1910.

Mr. MacMillan.

Load lbs. per sq. ft.	Deflection ins.	Remarks
405	9 32	Test continued July 25, 1910.
495	5/16	
540	11/32	
585	13/32	
626	1/2	
704	5/8	
741	3/4	During the placing of the last layer of bags very fine cracks were discovered for the first time, running through the center of floor slab both ways.
775	13/16	Cracks increasing slightly.
837	1 1/16	Cracks open entirely across the center of bottom of slab, no signs of cracks or spawling of concrete at head of column.
900	2	Test concluded at this point as far as loading was concerned. Large cracks nearly 1/8 in. entirely across bottom of slab the short way and half way from north end to center long way.

The slab was left loaded until July 28, the deflection increasing to 2 13/16 ins. On July 28, the load was removed and the slab still remains in position. It is the intention to break this up later.

The floor slab in a building at Buffalo was designed for a superimposed load of 150 lbs. per sq. ft. and the thickness of the slab was 8 ins. The column heads were designed large enough to reduce the shearing stress caused by the total load on the panel to a maximum of 80 lbs. per sq. in. The slab was designed for 150 lbs. per sq. ft. but the loads actually used, while they equalled that amount over the entire area, were concentrated so that in places the load amounted to 350 lbs. per sq. ft. One bay of the floor was tested with a load of 120,000 lbs. on one panel 20 ft. 6 ins. square, showing a deflection of 3/32 in. This load consisted of cast iron cylinders, fly wheels and other automobile parts, and was placed with the view of eliminating possible arch action. There were apparently no signs of cracks, and on that showing the floor has been loaded in the way intended. The working load consists mostly of bolts and small metal parts which are piled in racks, bringing concentrations that amount to fully 350 lbs. per sq. ft. on parts of the slab.

Mr. MacMillan. A careful examination made some time after the building was erected and the floor loaded with its regular working load over a large number of bays indicated in a few instances very fine hair cracks approximately similar to those found by Mr. Lord. These cracks occurred in the granolithic finish, being so fine that it required very careful observation to detect them, and in my opinion were no more than would naturally occur in any concrete member where the reinforcement was stressed to its working load.

The above floor was designed in accordance with the method explained at the Chicago Convention in February, 1910.* While a panel after this method was designed for a bending moment of $WL^2/25$ in the center of the slab, it is apparent that owing to the long laps of the reinforcing bars over the column heads, the reinforcement there is approximately double the amount in any other portion of the slab, and that as far as the reinforcement is concerned, the slab has a safe bending moment over the columns of $WL^2/25$. With the very large column heads it has been my practice to use, the concrete is in all probability not stressed excessively at the point of maximum negative moment. It would be of great interest to engineers in general if in future tests especial attention were given to stresses occurring in outside panels, and particularly the stresses caused in the wall columns by the deflection of such panels. This information would fill a very real need.

There is another point of interest to owners and that is the speed of erection. A building 100 ft. wide and 256 ft. long, in Lowell, Mass., was started in June, 1910, and on December 13, 1910, the brick work was within a week of completion. That is, a building with 7 acres of floor space was erected in a little over 5 months, so that as a speedy and economical proposition, it is probably unique. In this particular building the corrugated iron forms were used. These forms served to break up the dead flatness of the ordinary flat slab floor and improved, if anything, the general appearance of the ceiling.

Mr. Fox. MR. BENJAMIN FOX.—What value was used for the bending moment at the center of the span, in the case of the floor tested?

* A Simple Method of Computing the Strength of Flat Reinforced Concrete Plates, by Angus B. MacMillan, *Proceedings*, Vol. VI, page 248.—ED.

Mr. A. R. Lord.—A bending moment of about $WL^2/29$ at the center was used considering all of the bands and one-half way around the column. Mr. Lord.

Mr. W. B. Phillips.—In the floor under test, was reinforcement provided at the center of span, at right angles to that connecting two columns at the side of the panel? I believe some reinforcement should be provided between the columns in order to take care of the reverse moment, eliminating the cracks to which Mr. Lord refers. Mr. Phillips.

Mr. A. N. Talbot.—In so far as I know such reinforcement is not used in any of the girderless floors. Mr. Talbot.

The cracks referred to are minute hair cracks which when the load is removed will close up so as not to be visible. They are the same kind of cracks as appear in the ordinary concrete beam at stresses of 6,000 to 15,000 lbs. per sq. in. in the reinforcement. So far as any one knows, they are not detrimental to a structure, and are not of such a nature as to prevent the protection of the reinforcement by the concrete. We may expect at this point of reversal of stress that a crack will appear in the surface. The cracks may be visible but are not such as appear when the reinforcement has passed its yield point. They can be seen more readily by means of a magnifying glass, but are visible, however, without the glass, by very careful inspection.

As far as analysis is concerned it would seem to me more satisfactory to have some analytical basis for the design of any type of construction. It may be that an analysis of the very complex structure will give results higher—moments and calculated stresses higher—than will be developed in the structure; but even then there is an advantage in having a basis for our work. In the case under discussion and for the size of the capital given, the calculated stresses based upon an assumed distribution of load, (and variations in assumptions as to this distribution, as pointed out by Mr. Lord, will not greatly affect the results) give at the supports 1/12 to 1/14 as a coefficient of WL^2 and the center of the panel and half way between the columns, a bending moment about one-half as great. These coefficients do not differ much from coefficients for ordinary restrained beams. When we come to measure the stresses in such buildings, it is to be expected that

Mr. Talbot. the deformations and deflections will be less than those computed on any such assumption. We should expect to have the same difference in ordinary beam and girder construction.

There have been observed, no doubt, the low values obtained in load tests of floors. These result partly from the slab action of the horizontal beam which spreads the stresses over a considerably larger width than is counted on in the calculations, and also the arch action at the ends of the slabs or beams. Now, the bending moment coefficients in beam and floor construction and beam and girder construction are not reduced because it is felt that the stresses are less than they are calculated to be. The question is, in flat slab construction, shall coefficients of bending moments be reduced below those indicated by analysis if it is not known whether the stresses will be transmitted around the panels by the remainder of the construction. This is a question of considerable importance.

On the diagram of deflections there is an increase in the deflection at the middle of the panel when the load remained over night and the deformation at the supports became less. That is to be expected; these two go together. The decrease in deformation results from greater distribution of the stress over the panel. It is not that there is less total stress, but after time is given to such a plastic material as green concrete, the distribution of stress is effected over a greater length.

Mr. Lord deserves special credit for his work on this test. He exercised a great deal of care and displayed initiative and judgment and also much thoughtfulness in making provision in advance of the test. Credit also should be given to the Concrete Steel Products Company and the Leonard Construction Company of Chicago for their enterprise and public spirit in having the test made and in bearing the expense.

Mr Thompson. MR. SANFORD E. THOMPSON.—The test of the Minneapolis building described by Mr. Lord and made under the general supervision of Professor Talbot, represents one of the most important advance steps in reinforced concrete. It opens up an entirely new field, and if the methods adopted are followed by further similar tests on completed structures they will throw most valuable light on the action of reinforced concrete under working conditions and

which could be obtained in no other way. The beauty of the idea lies in the fact that the actual stresses can be determined without injuring the building. Load tests by ordinary methods, in which the deflections are measured, are practically worthless from the designer's standpoint unless the test is carried to destruction, and even then no information is given as to the stresses at working loads. Mr. Thompson.

Our Reinforced Concrete Committee can do no more valuable work than arranging for future tests of a similar nature, and I trust that all who are interested will write at once to the Chairman, Mr. A. E. Lindau.

The results from tests of this nature must be applied to actual design with careful judgment. One reason why so few failures have occurred in reinforced concrete construction is because our formulæ and our general practice in design do not take into account the arch action and effect of monolithic construction, so that our structures are really safer than the designs would indicate. For example, in the ordinary beam and slab floor, the actual stresses under a given load are certainly less than those for which the members were computed because of this arch action in monolithic construction. In view of the difficulty, even in first-class construction, of placing every bar exactly according to plan, and also the difficulty in practice of providing always a good quality of sand and gravel, this added factor of safety is of infinite value. When, therefore, we do obtain our results from stresses in actual buildings, we must be careful not to increase our fundamental units of stress to conform too exactly to the actual stresses, but, instead should use the tests in the completed building to enable us to balance our stresses more exactly and provide a uniform factor of safety at all points in the structure.

The results of the Minneapolis test show a well balanced design and yet the stresses at a working load of 225 lbs. per sq. ft. come too near the limit of safety to say that the design should be used as an example for future buildings which are to be built on a safe and conservative basis. In other words, we must and ought to expect, as I have indicated, that practical tests of well constructed buildings will show lower stresses than good practice will authorize in design. I do not mean to say by this that we should

Mr. Thompson. allow a "factor of ignorance," but even with the best of supervision there are so many points in reinforced concrete construction that tend to occasional points of weakness because of the field nature of the work, that we must be further on the safe side than where, as in steel construction, the final arrangement is open to view.

This criticism does not in any way reflect upon the value of the tests from the scientific or practical standpoint, but it is made merely to suggest the fact that while this building will undoubtedly meet all requirements, the stresses steer too closely to the danger limit to permit entire approval of the design as an example for future construction.

As a result of extended study of the theory and comparison of different types of design, it appears that the most economical arrangement of reinforcement at the column is to place two layers, consisting of the bars in the two diagonal directions, in the top of the slab and the two rectangular layers in the bottom of the slab. The bars in the bottom, which assist in taking the compression, may be less in quantity, usually about half, for the greatest economy, than in the top of the slab. With this distribution a slab of minimum thickness is provided (1) because the reinforcement in the bottom aids in taking compression and hence reduces the stress in the center itself; (2) the center of gravity of the tension reinforcement is brought nearer the surface than if four tension layers are provided; and (3) the center of compression of the concrete and compression reinforcement is brought nearer the bottom surface. By this means, the depth to tension reinforcement and also the distance between centers of tension and compression are made as great as possible for a given thickness of slab and conversely, the same reinforcement gives the thinnest slab consistent with good design.

Mr. Lord. MR. LORD.—In connection with Mr. Thompson's remarks on the question of the allowance to be made for unknown factors, I may mention again one feature of the test. When undertaking the test I reserved the right to pick my own spot for the test, and selected what was thought to be the weakest spot which could be expected to occur in any well constructed building. Bulkheads

were very numerous, being present in every panel tested except one. The concrete on this floor was only 40 days old at the beginning of the test. In digging for the rods to attach our instruments, we found a variation of ½ in. in the up-and-down dimension of the rod in the slab (the embedment); and in general the conditions were unfavorable to a high showing of strength. Mr. Lord.

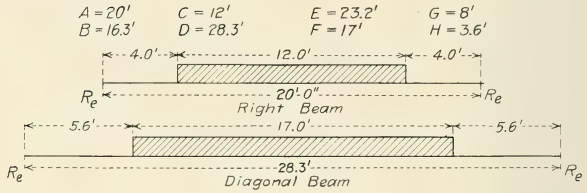
In dealing with a new type of construction we should not be governed too rigidly by previous construction. In beam and girder construction we have a weakness not found in the one tested; that is the matter of junctions. The points where construction is temporarily discontinued are more numerous, and the time between pourings greater, and hence the effect is more marked. We should work, it seems to me, on the basis of designing the flat slab by what appears right and reasonable, and modify our beam and girder construction in the same way if it seems advisable.

Mr. W. H. Ham.—A method of flat slab floor construction somewhat different from that described by Mr. Lord, is one in which the reinforcement is arranged in bands or belts in both right line and diagonal line between columns and a second layer of reinforcement is placed at the top of the slab in the diagonal lines. Mr. Ham.

The diagram, Fig. 1, shows the assumptions made in dividing the stress between the different beams and the values given are for a panel of 20 ft. span. The area at the center, indicated by wide cross-hatching, is assumed to be carried by the two diagonal beams, the side areas, indicated by close cross-hatching, by the right beams. This assumption is the same as saying that the stress will reach the column by the shortest distance.

I think, therefore, it is fair to assume that the loads, both dead and live, for the center area will be carried to the column by a belt or band, marked *G*, and that the loads on this area will develop stresses in the slab *G* which can be determined by calculating for stresses due to moments figured for a beam loaded a portion of its length with a uniform load.

If the beam is continuous over a number of supports, the bending moments will be reduced accordingly. Assuming the continuity reduces the bending moment over the support in proportion of 1/8 to 1/10 of the span and the moment at the center of the



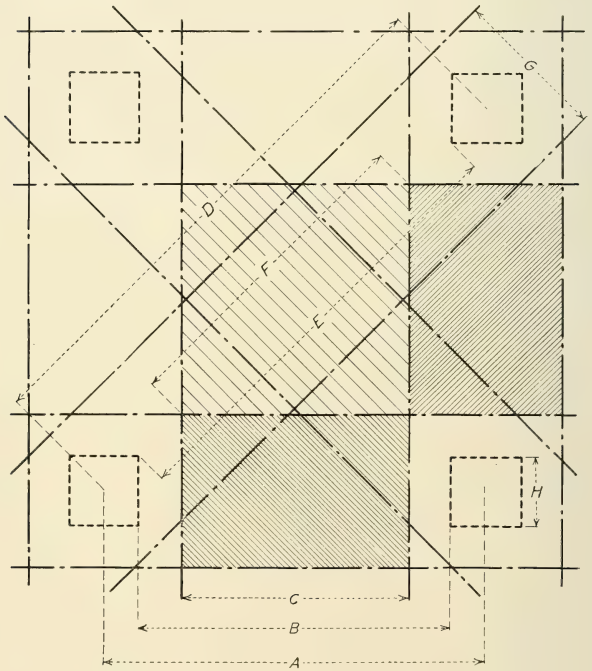


FIG. 1.—DIAGRAM SHOWING SCHEME OF FLAT SLAB DESIGN.

span as ½ that at the support, the equation for the diagonal beam bending moment would be stated by the following formula: Mr. Ham.

$$M_D = [(R_e \times 14.2) - (R_e \times 4.5)] \times 12 \times \tfrac{8}{20}$$

where R_e = End Reaction.
and M_D = Bending Moment at center of diagonal beam.

Assuming likewise the side area to be carried by the right beam the bending moment is expressed by the following formula:

$$M_R = [(R_e \times 10) - (R_e \times 3)] \times 12 \times \tfrac{8}{20}$$

M_R = Bending Moment at center of right beam.

Having found the bending moment, the amount of reinforcement required is found by designing the band of a width equal to G, as a rectangular beam whose span length is the diagonal distance between columns or the right distance between columns for diagonal and right beam respectively.

The remaining load over the column can fairly be assumed as transmitted to the column by means of a cantilever. It will bring tensile stress into the top reinforcement only. If the reinforcement of the right line beam is lapped across the column the width of the belt, or in other words to the point of contraflecture there will be an excess of reinforcement beyond the edge of the cap sufficient to resist the stress due to this cantilever action.

The stresses, therefore, are all cared for except the negative tensile stress due to the load of the center area, transmitted by the diagonal beams. This will require a mat of reinforcement at the top of column, the length reaching to the point of contraflecture in both directions. The stress in this mat of reinforcement will be equivalent to the stress at the center of the diagonal beam provided the depth of the column cap at the center is double the depth of the slab at the center of the span. The critical point, however, will be at the edge of the cap. At this point, the bending moment has been reduced and if the cap is of the proper size will not exceed ½ the negative bending moment at the center of the column. Thus, the top mat of reinforcement would have the same number and size of bars as the diagonal belt.

A set of calculations for a flat slab panel 20 ft. square, Fig. 1,

Mr. Ham. with slab 8 ins. thick, based on the above assumptions, would be as follows:

Loads.

Live	= 150	lbs. per sq. ft.
Slab	= 100	" "
2 ins. cinders	= 16	" "
$1\frac{1}{4}$ ins. wood	= 5	" "
Total	= 271	lbs. per sq. ft.

Right Beam. Load $= 8 \times 12 \times 271 = 26{,}000$ lbs.

$$R_e = \frac{26{,}000}{2} = 13{,}000 \text{ lbs.}$$

$$M = \frac{[(13{,}000 \times 10) - (13{,}000 \times 3)] \times 12 \times 8}{20} = 436{,}500 \text{ in.-lbs.}$$

$$R = \frac{436{,}500}{96 \times 7 \times 7} = 93 \quad P = .68 \text{ per cent.}$$

Area of reinforcement required $= 96 \times 7 \times .0068 = 4.57$ sq. ins.
Use $\frac{5}{8}$-in. round rods, 6.5 ins. on centers $= .57$ sq. ins.

Reinforcement over top of column. Required 4.98 sq. ins.
Use $\frac{5}{8}$-in. round rods, 5.5 ins. on centers, 8.5 ft. long.

Diagonal Beam. Load $= 12 \times 12 \times 271 = 39{,}000$ lbs.
$\frac{1}{2}$ load each way $= 19{,}500$ lbs.

$$R_e = \frac{19{,}500}{2} = 9{,}750 \text{ lbs.}$$

$$M = \frac{[(9{,}750 \times 14.2) - (9{,}750 \times 4.25)] \times 12 \times 8}{20} = 463{,}000 \text{ in.-lbs.}$$

$$R = \frac{463{,}000}{96 \times 7 \times 7} = 102P = .74 \text{ per cent.}$$

Area of reinforcement required $= 96 \times 7 \times .0074 = 4.98$ sq. ins.
Use $\frac{5}{8}$-in. round rods, 5.5 ins. on centers $= .67$ sq. in.

This method of analysis of the flat slab stresses seems to the writer to have a good many points of merit, among them being the following:

1. Slabs of different size and of different conditions of continuity can be readily computed.

2. The standard beam formulas which are thoroughly recognized by engineers can be used in the computation.

3. The critical point, namely near the edge of the column cap can be treated independently from the column itself, the reinforcement for the columns often being varied on account of the

loads from floors above and sometimes being increased or decreased to vary the size of the column in the design. Mr. Ham.

4. The stresses are more nearly fixed by the will of the designer than in the method of reinforcing tested by Mr. Lord.

5. Smaller rods are used for the negative tension than is usual with the other systems of reinforcement. These smaller rods can be placed more accurately than the larger rods of the columns which are sometimes bent out and there is also less waste due to lap.

6. The designer by varying the widths of the bands (different widths for diagonal and right beam bands if desired) can keep the reinforcement at almost exactly the same percentage in both diagonal and right beams, thus making the design very elastic in its application.

7. The reinforcement which seems by virtue of the result of tests reported by Mr. Lord to be in excess of the requirements, will, in my judgment, be required to resist stress due to accidental loads which will surely come to a building having as long a life as one of concrete, and any such excess reinforcement will tend to reduce to a minimum the danger of cracks in the floor.

8. The designer may take advantage of the continuity in one direction and reinforce properly for simple beam action in the other direction, as in the end panels.

9. Hooping for concrete columns may be carried through the entire head of the column shaft without the large excess of reinforcement caused by bending out additional rods or the weakening of the column due to bending out a portion of the longitudinal reinforcement. Structional steel cores can be used readily with this type of reinforcement.

10. The tendency to split the column by tension of rods which are bent out from the column is eliminated. This tendency to split the column is resisted by the top hoops in the systems similar to the one tested by Mr. Lord and this hooping at the top of the column is subject to the most extremes of heat in case of fire. The system here described does not require hooping of columns at this point.

This system of placing steel first came to the writer's attention in connection with the Quincy Cold Storage Market Building which

Mr. Ham. was inspected several times during construction. Fig. 2 shows the reinforcement in place. This was, as far as I know, the first use made of this method, and it is believed that Mr. J. R. Worcester is responsible for the design of this system of placing of reinforcement, although the writer is not familiar with the assumptions made by Mr. Worcester. It is now being used quite extensively in various buildings and all, so far as the writer can learn, have been successful.

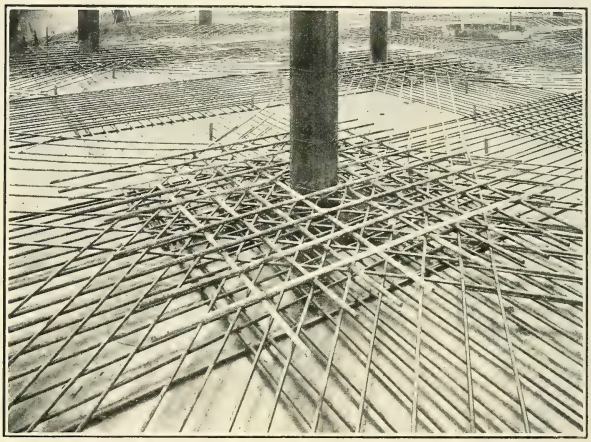


FIG. 2.—REINFORCEMENT FOR FLAT SLAB FLOOR IN PLACE.

The writer agrees with Mr. Lord that the low stresses shown in the test were due to arch action and desires to express the opinion that this arch action should only be considered in connection with increasing the factor of safety and should not be figured in the design. Concentrated loads may overcome this arch action and develop permanent injury to the slab if the reinforcement supplied for tension is not sufficient to resist the stresses without the arch action. This is fundamental and this Association should look toward the revising of live load requirements in various building codes if they are excessive, but should religiously maintain the

high standard of engineering which has so long demanded factors of safety, with the unknown quantities eliminated from the calculations. Mr. Ham.

The wonderful advancement of concrete construction in the last 10 years is guarantee enough for those interested that the future of this type of construction does not need the false economy of reducing the factors of safety by depending upon the arch action of beams or flat slabs.

Mr. G. H. Brazer.—With regard to flat slab construction, I might say that the placing of the concrete had always struck me as being a very difficult proposition. When having an opportunity, however, to see the concrete deposited, I was surprised to observe with what readiness the concrete settled into position around the network of rods over the columns. The structure referred to was that shown in Fig. 2, and is the Quincy Market Cold Storage Warehouse. On these floors the diagonal rods were straight and laid at the bottom of the slab. The rods on the line of the columns were turned up over the column heads for negative reinforcement. To provide for negative reinforcement in the diagonal direction, larger rods were placed near the top of the slab which had the same cross sectional area as the diagonal rods; the object in using the larger sized rods being to reduce the number. Mr. Brazer.

It always seemed to me a good scheme to keep some of the reinforcement at the bottom of the slab around the column, and the diagonal reinforcement seems best adapted for this purpose.

Mr. Ham.—I wish to emphasize the point brought out by Mr. Brazer. The minimum spacing between the rods was 6 ins., but the diagonal view taken by the camera shows them more congested than they are. The rods are at right angles and at 45 deg. with each other, but over the column head the smallest opening is 6 ins. either triangular or square, which would allow any aggregate of reasonable size to be used. Mr. Ham.

The rods are held away from the forms by means of sockets and are so well supported and wired that heavy concrete wheelbarrows can be run on them without displacing the rods.

Mr. Brazer.—The proportions of the concrete were 1 : 2 : 4, and nothing larger than ¾-in. stone was used. The consistency was wet but not sloppy. The concrete was well sliced around the rods, Mr. Brazer.

Mr. Brazer. and we felt well satisfied with the result. The upper layers of rods were in contact where they crossed, but there is a slight separation between the lower horizontal rods and the upper tiers which cross over the column.

In regard to the cost of the flat slab in comparison with the beam and slab construction—they are very nearly equal. Due to the fact that the flat slab construction usually is of considerably less depth than the beam construction, one naturally expects to find the cost of the former greater than the latter, but as is generally known, the centering for concrete work is a large percentage of the total cost, and it is evident that the simple centering required for the flat slab is a great deal cheaper than that for the beam and slab construction.

With regard to the use of flat slab construction, I would say that it is advantageously used where a smooth ceiling is required, and also in a building where the greatest number of floors are to be obtained in a limited height of building. In storage warehouses, frequently the materials are piled right up to the under side of the floor above, and in these cases, one can readily see that every square foot of space is being used, and none wasted, as would be the case if the ceiling above were cut up with beams and girders.

Mr. Mensch. MR. L. J. MENSCH (By letter).—The test of a flat slab floor as described by Mr. Lord is certainly the most thorough test that ever came to the writer's knowledge and the concerns who paid for such a test certainly deserve the thanks of the engineering profession.

The test clearly brought out that the smallest bending moment occurs in the center of the slab; that the bending moment in the center of the cross bands is probably 30 per cent. larger than in the center of the slab, and that the bending moment around the capital is the largest moment in the flat slab of this type, and that this maximum moment at the capital really governs the design of the slab.

It also clearly demonstrates that in a continuous slab the maximum stresses are around the sides of the squares at the top of the slab, which fully proves Grashof's theory as shown in the writer's paper* on same.

*The Calculation of Square Plates Supported by Four Columns. See p. 205. —ED.

Mr. Mensch.

It is to be regretted that the test did not extend to the columns and that the deformations and stresses in the columns were not obtained. This test also showed that for this particular case the stresses in the outside panels are about 25 per cent. more than in the inside panel.

Analyzing the slab according to the method of the writer, we can find that the moment of inertia of a column 30 ins. around = $(\pi/64)\ 30^4 = 39{,}000$ in.^{4}; the moment of inertia of the slab = $226.5\ (9.1875^3/12) = 14{,}600$ in.^{4}; and assuming the height of the floors 12 ft.

$$n = \frac{12}{19}\,\frac{14600}{39000} = .237.$$

This value of n introduced in equation (21) of the writer's paper gives

$$M_B = \frac{pd^3}{17.4} \times \frac{1}{1 + \left(\frac{.237}{2}\right)} = \frac{pd^3}{19.46}$$

and

$$M_C = \frac{pd^3}{11.6} - \frac{pd^3}{19.46} = \frac{pd^3}{28.7}$$

and on account of Poisson's Ratio $M_B = pd^3/27.7$ and $M_C = pd^3/41$.

If we assume that the bending moment M_C is distributed over the whole length of d we can say that the bending moment per lineal foot in the center of the diagonal band for which the slab is to be figured $= pd^2/41$ while in the center of the cross band we have not the favorable influence of two equal bending moments acting under 90 deg., and therefore the bending moment per lineal foot will be only $= pd^2/28.7$.

The value of M_B above given refers to two columns or the bending moment for one column $M_B = pd^3/55.4$. This bending moment acts both in the direction of the X axis and in the direction of the Y axis and for each direction the bending moment is $pd^3/55.4$.

It is probably going to the utmost if we assume that the width of the slab acting at the column head $= \frac{1}{2}d$, which would require the reinforcing rods also to be placed in bands of a width of $\frac{1}{2}d$ and the bending moment per lineal foot of these bands $= pd^2/27.7$.

Mr. Mensch. In the case of the test panel the width assumed was $.4d$, which gives a bending moment per lineal foot = $(pd^3/55.4) \div 4d = pd^2/22.16$. Assuming $p = 337.5$ lbs. per sq. ft., the bending moment per foot = 5,500 ft. lbs. The reinforcement over the column head is given as 75 per cent. greater than for the center of the panel, or about .44 sq. ins. per lin. ft. Assuming the effective depth of the slab = 7.5 ins., we find the stresses in the rods from the equation $5{,}500 = (7.5/12)\ .87 \times .44 \times S$, or $S = 23{,}000$. For a total load of 462.5 lbs. per sq. ft. we would obtain in a similar way a stress of 28,400 lbs. per sq. ft., which closely corresponds to the stresses found by Mr. Lord. If anybody doubts that such high bending moments exist over the columns, I wish to call attention to the fact that according to Grashof there exists negative bending moments over the whole sides of the squares and this bending moment amounts to $pd^2/26.4$ per lin. ft. and it is clear that we have to place at least as many rods over the columns if we omit to do it at the sides of the squares.

The test also proved that for continuous panels a strict adherence to Grashof's analysis would have prevented the cracks mentioned in the paper. The test clearly showed that practically at no time were the stresses and loads proportionate. It also showed the great influence of the tensile strength of the concrete on the stresses in the reinforcing rods. Stresses of 6,000 lbs. as found in the center rods would correspond to a bending moment of $pd^2/140$ which is clearly impossible. From the deflections and stresses for the various loads we can also infer that straight planes do not remain straight under bending and that the modulus of elasticity varies greatly under different loads.

From the writer's experience with telegraph poles he would estimate the modulus of elasticity at the beginning of the test at 1,500,000 lbs. and the modulus of elasticity at the load of 350 lbs. at 1,000,000 lbs. and emphatically disagrees that there were such high compressive stresses as 1,000 lbs. per sq. in. in the concrete, as Mr. Lord infers.

THE CALCULATION OF REINFORCED CONCRETE FLAT PLATES SUPPORTED BY FOUR COLUMNS.

By L. J. Mensch.*

The interest taken in the last few years in girderless floor construction, which was first introduced to any large extent by the author at the plant of Proctor and Gamble, Armourdale, Kans., in 1903, for the high loads of 2,500 lbs. per sq. ft., and the many discussions in regard to the proper calculation of such construction, caused the author to look through the books treating of the elastic theory. As far as the research allowed, it was found that the subject was solved by Professor Winkler in his book, *Lehre von der Elasticität und Festigkeit,* Prag, 1867, and by Professor Grashof in his book, *Theorie der Elasticität und Festigkeit,* Berlin, 1878.

Inasmuch as the results are rather surprising, the author will compare the theory of plates supported by four columns with the theory, practice and tests of round and square plates.

In the following discussion let

d = span, taken in feet for the bending moment, and in inches for the deflections;

p = uniform load, in lbs. per sq. ft. for bending moments, and in lbs. per sq. in. for deflections;

h = thickness of plate in inches;

E = the modulus of elasticity.

The theory employed by Winkler and Grashof is similar to that applied in developing the formulae for the deflection of a common beam, which is, however, rather too complicated for presentation.

Case I.—Round plate uniformly loaded, and supported at its circumference.

*Consulting Engineer, Chicago. Ill.

For this case Grashof gives the maximum bending moment in the center =

$$\frac{3}{8}\,\frac{(m-1)(3m-1)}{m^2}\,\frac{d^2p}{24} \quad \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

and the greatest deflection =

$$\frac{3}{256}\,\frac{(m-1)(5m+1)}{m^2}\,\frac{pd^4}{Eh^3} \quad \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

in which equations $\frac{1}{m}$ represents the so-called Poisson's ratio which we may assume nearly as $\frac{3}{10}$. For those not familiar with the signification of Poisson's ratio, I wish to state, that if two forces are acting on a plate element at right angles, elongations in each direction are produced, which are $\frac{1}{m} = \frac{3}{10}$ times less than the amount of elongations which would be produced if only forces in one direction be acting.

Introducing Poisson's ratio in the above equations the maximum bending moment =

$$.708\,\frac{d^2p}{24} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

and the greatest deflection =

$$.0435\,\frac{d^4p}{Eh^3} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

If we consider the round plate like a common beam uniformly supported at its circumference, and assume that the plate breaks along the line of the diameter through the center, as shown in Fig. 1, the distance of the center of gravity of the reactions from $AB = \frac{d}{\pi}$ and the distance of the center of gravity of the uniform load from $AB = \frac{2}{3}\,\frac{d}{\pi}$.

The bending moment at

$$AB = \left(\frac{1}{2}\,\frac{d^2\pi}{4}\,p\,\frac{d}{\pi}\right) - \left(\frac{1}{2}\,\frac{d^2\pi}{4}\,p\,\frac{2d}{3\pi}\right) = \frac{d^3p}{24},$$

or the moment per lineal foot of $AB = \frac{d^2p}{24}$. On account of Poisson's ratio, this bending moment will be diminished by 3/10, and will be $= .7\,\frac{d^2p}{24}$, which perfectly agrees with the result given

by Grashof. Professor C. Bach tested round plates, made of cast iron and of steel, and found that, as long as the plate was uniformly supported at its circumference, the strength of such plates perfectly corresponds to the above formulae.

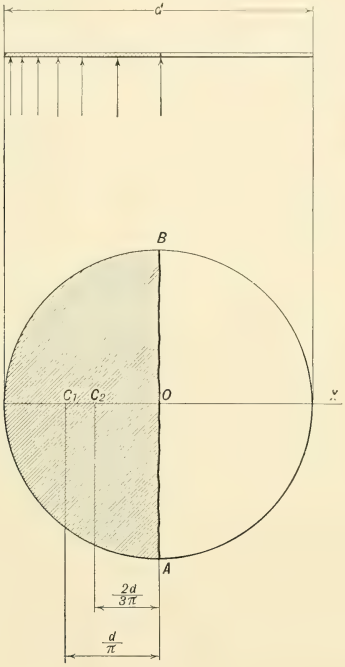


FIG. 1.

Case II.—Round plate, uniformly loaded and fixed at the circumference.

For this case Grashof gives the following formulae: The bending moment in the direction x at any point distant x from the origin =

$$\frac{3}{8}\,\frac{(m^2-1)}{m^2}\,\frac{p}{6}\left(\frac{d^2}{4}-3x^2\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (5)$$

For $x = 0$ – the bending moment in the center =

$$\frac{3}{8}\,\frac{m^2-1}{m^2}\,\frac{pd^2}{24} \quad \ldots\ldots (6)$$

For $x = \frac{d}{2}$ the maximum bending moment occurs at the circumference and =

$$\frac{3}{4}\,\frac{m^2-1}{m^2}\,\frac{pd^2}{24} \quad \ldots\ldots (7)$$

and the greatest deflection =

$$\frac{3}{16}\,\frac{m^2-1}{m^2}\,\frac{pd^4}{16h^3} \quad \ldots\ldots (8)$$

Introducing again Poisson's ratio, we have

$$M \text{ max.} = .6824\,\frac{pd^2}{24} \quad \ldots\ldots (9)$$

$$\text{Deflection at center} = .0107\,\frac{pd^4}{Eh^3} \quad \ldots\ldots (10)$$

whereof we can find that the fixature at the ends does not materially increase the strength of the plate. It, however, decreases the deflection to ¼ of that of a plate freely supported. We can find by comparing equations (6) and (7) that the stress in the center is just one-half of that at the circumference. Also this formula was tested by Professor Bach for its accuracy, and he found that it perfectly agrees with the tests.

Case III.—Plate supported by a great number of supports, arranged in corners of squares.

Professor Grashof gives the bending moment at any point distant x from the origin=

$$\frac{1}{2}\,\frac{(m^2-1)}{m^2}\,\frac{p}{6}\left(\frac{d^2}{4} - 3x^2\right) \quad \ldots\ldots (11)$$

The bending moment in the center =

$$\frac{1}{2}\cdot\frac{(m^2-1)}{m^2}\cdot\frac{pd^2}{24} \quad \ldots\ldots (12)$$

The maximum bending moment at the sides of the squares =

$$\frac{(m^2-1)}{m^2}\,\frac{pd^2}{24} \quad \ldots\ldots (13)$$

and the greatest deflection =

$$\frac{1}{2}\,\frac{(m^2-1)}{m^2}\,\frac{pd^4}{16Eh^3} \quad \ldots\ldots (14)$$

Or, if we introduce Poisson's ratio

$$M \text{ center} = .455 \frac{pd^2}{24} = \frac{pd^2}{52.8} \quad \ldots\ldots\ldots\ldots\ldots (15)$$

$$M \text{ max.} = \frac{pd^2}{26.4} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (16)$$

The greatest deflection =

$$.0284 \frac{pd^4}{Eh^3} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (17)$$

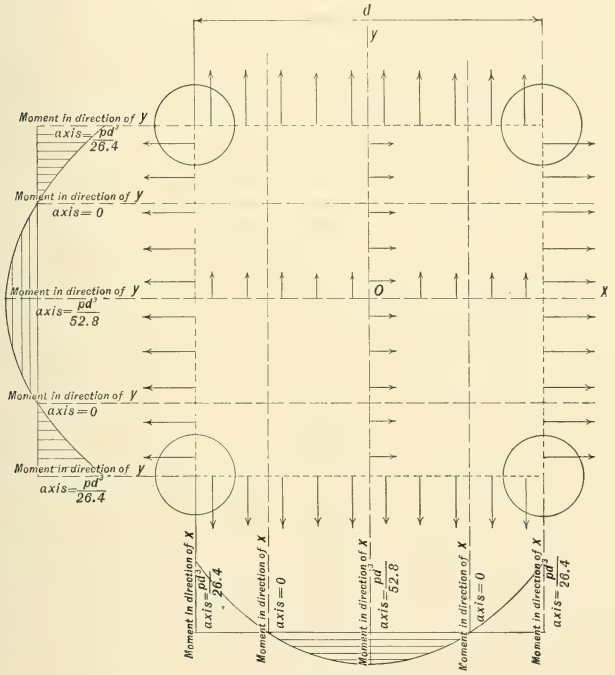

d
y
Moment in direction of y axis = pd²/26.4
Moment in direction of y axis = 0
Moment in direction of y axis = pd²/52.8
Moment in direction of y axis = 0
Moment in direction of y axis = pd²/26.4
O
X
Moment in direction of X axis = pd²/26.4
Moment in direction of X axis = 0
Moment in direction of X axis = pd²/52.8
Moment in direction of X axis = 0
Moment in direction of X axis = pd²/26.4

FIG. 2.

If we compare these equations with the results of the round plate, fixed at the end, we find that the distribution of stresses is nearly identical and that the maximum bending moment and the

maximum deflection is $\frac{2}{3}$ of those of a circular plate, having a diameter equal to the diagonal of the square.

The distribution of stresses is shown in Fig. 2, requiring in concrete slabs two lines of reinforcement at the underside of the slab, running at right angles, and extending the full width of the

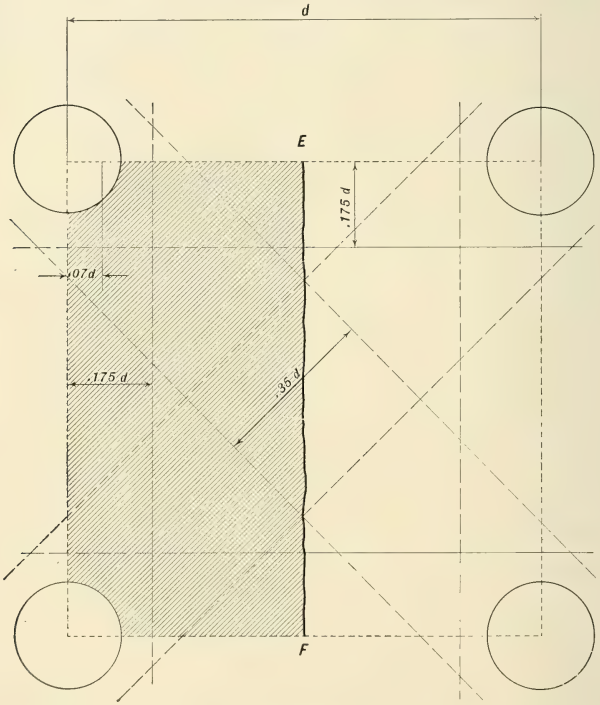


FIG. 3.

squares, and shorter and heavier reinforcements at the top of the slabs, over the sides of the squares, and running at right angles to the sides; hence, although the bending moment per lineal foot is comparatively small, the sum of all bending moments, which govern the amount of reinforcement, is larger than expected.

We shall try, also for this case, to solve the problem by the ordinary theory in a similar way as we did for the case of a round plate. If the plate is only supported by four columns without rigid connections with these columns, as shown in Fig. 3, we can assume that the center of gravity of the reactions is distant $.07d$ from the center of the columns, and the bending moment at

$$EF = \frac{d^2}{2} \; p \left(\frac{d}{2} - .07d \right) - \frac{d^2}{2} \; p \; \frac{d}{4} = .090 d^3 p.$$

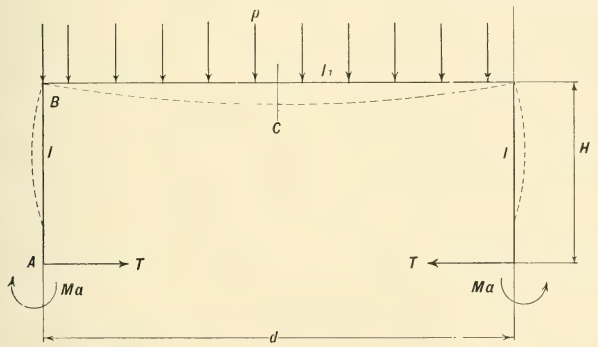


FIG. 4.

Or, inasmuch as the capitals of the columns take up about 4 per cent. of the total area, this moment =

$$.0864 \, d^3 p = \frac{d^3 p}{11.6}.$$

On account of Poisson's ratio this value will reduce to

$$\frac{d^3 p}{16.5} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (18)$$

This value may be considered acceptable, if we compare it with the formulae for the bending moment of a square plate supported at the four sides, which is generally taken at

$$\frac{pd^2}{20} \text{ or } \frac{pd^2}{24}.$$

In case of a building these plates are supported by columns and rigidly connected, and we can assume that the four columns and the plate form a monolithic frame, and that the deflection of the plate will cause also a deflection in the columns and, inversely, that the deflection of the column will retard the deflection of the slab, as shown in Fig. 4.

Where the slab is uniformly loaded and would have a bending moment in the center $= \frac{pd^2}{8}$, if freely supported, we can find by the application of the theory of least work, the bending moment at B

$$M_B = \frac{pd^2}{12} \frac{1}{1 + \frac{n}{2}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (19)$$

where

$$n = \frac{H}{d} \frac{I_1}{I} \quad \ldots\ldots\ldots\ldots\ldots\ldots (20)$$

and wherein I_1 is the moment of inertia of the plate of the width d, and I the moment of inertia of *One* column.

In our case the moment in the center $= \frac{d^2p}{11.6}$

and the corresponding moment

$$M_B = \frac{pd^3}{17.4} \times \frac{1}{1 + \frac{1}{2\ n}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (21)$$

The value of n is in most cases of practice nearly 1, hence

$$M_B = \frac{pd^3}{26.1}$$

and the moment in the center

$$M_C = \frac{pd^3}{21.6}.$$

Or on account of Poisson's ratio

$$M_C = \frac{pd^3}{30.9} \quad \ldots\ldots\ldots\ldots\ldots\ldots (22)$$

and

$$M_B = \frac{pd^3}{37.3} \quad \ldots\ldots\ldots\ldots\ldots\ldots (23)$$

If the plate is actually a single square and supported by four columns, then M_B is to be divided by two, to obtain the value for

one column. However, if a number of plates in one row are loaded, then the bending moment at the column would be the value as given in (23), if we neglect the continuity of the slabs in one direction, which is evidently figuring too safely, and we can place the moment at one column

$$M_B = \frac{pd^3}{74.6} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (24)$$

for every case.

As a rule the columns run through several stories, which increases the stiffness of the columns. Assuming, therefore, $n = \frac{1}{2}$, in equation (21),

$$M_B = \frac{pd^3}{31.1} \text{ and } M_C = \frac{pd^3}{34.6},$$

Poisson's ratio considered.

On the other hand, if panels are alternately loaded, this favorable point will be lost, and the values of (22) and (24) should be acceptable to conservative designers.

If the columns are very heavy in comparison with the slabs, so that the slabs are immovably fixed at the columns, like in a continuous plate on many supports, n vanishes, and

$$M_B = \frac{pd^3}{17.4} \text{ and } M_C = \frac{pd^3}{34.8},$$

or on account of Poisson's ratio

$$M_C = \frac{pd^3}{49.8} \text{ and } M_B = \frac{pd^3}{24.9},$$

which agrees with the values found by Grashof, and proves that the later introduction of Poisson's ratio, when obtaining the principal stresses is admissible, a point, which may allow the solution of problems, not yet solved.

Both according to Grashof's figuring, and the method here presented, a reinforced concrete slab should be reinforced in directions parallel to the sides of the squares. On account of the great shear values at the columns it is, however, desirable to run portion of the reinforcement in the direction of the diagonals, and we can count one diagonal band as corresponding to the X axis and the other band as belonging to the Y axis. The author gave in his pocket book the rule to make the width of the band of reinforcement equal to $.35d$. Then the width of the bands in the direc-

tion of the X or Y axis is $.7d$, and the moment per lineal foot of the band of reinforcement is

$$\frac{pd^3}{30.9} : .7d = \frac{pd^2}{21.6} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (25)$$

At the column the width of reinforcement in the direction of the X axis is only $.35d$, and therefore, the moment per lineal foot =

$$\frac{pd^3}{74.6} : .35d = \frac{pd^2}{26.1} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (26)$$

We can evolve the following rule for the figuring of plates supported by columns in the corners of squares:

For the dead loads the moment in the center of the slab per lineal foot of band is to be taken as =

$$\frac{Wd^2}{52.8} .7d^2 = \frac{Wd^2}{36}.$$

Over the entire sides of the squares is to be taken as =

$$\frac{Wd^2}{26.4} \text{ per lineal foot.}$$

For the live loads the moment in the center per lineal foot of bands of a width of $.35d$ is to be taken as = $\frac{pd^2}{20}$.

It is not advisable to assume a smaller bending moment for the top reinforcement over the columns than $\frac{pd^2}{20}$, as there are to be placed four layers of rods which diminishes the effective depth of the slab.

As a rule the negative moments over the columns are taken care of by bending the bottom rods up, near the columns, which often produces eight layers of rods, instead of four, and only one half of the rods, and even less, can be counted as effective to take up the negative moments. Safe practice requires special top rods as advocated above.

One designer places rings around the columns, which are invariably near the underside of the slab. They are useless there, but would be effective if placed at the very top of the slabs. The columns proper must be figured for a bending moment = $\frac{pd^3}{74.6}$, when p represents the live load in lbs. per sq. ft. As a rule the additional strain on the columns is very small.

In the case of plates, supported along the sides of squares, Grashof found difficulties in solving the problem but thinks that at a safe guess the greatest moment over the sides of the squares may be taken at $\frac{pd^2}{24}$ and that the moment in the center may be taken at $\frac{pd^2}{48}$ per lineal foot for continuous plates.

We shall try to solve the problem for non-continuous plates by

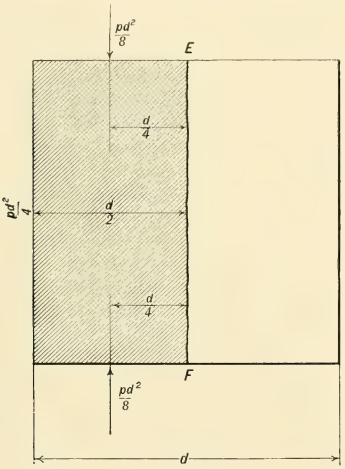


FIG. 5.

the author's method. We assume, see Fig. 5, that the reactions are uniformly distributed over the sides of the square and equal $\frac{pd^2}{4}$ for each side.

The moment at

$$EF = \left(\frac{pd^2}{4}\,\frac{d}{2}\right) + \left(\frac{2.pd^2}{8}\,\frac{d}{4}\right) - \left(\frac{pd^2}{2}\,\frac{d}{4}\right) = \frac{pd^3}{16}$$

and on account Poisson's ratio $MC = \frac{pd^3}{23}$, which corresponds closely to practice and tests made by Prof. Bach on cast iron plates, and tests made in Paris on reinforced concrete slabs of the "Palais des Costumes."

ANALYSIS OF RESULTS OF LOAD TESTS ON PANELS OF REINFORCED CONCRETE BUILDINGS.

By Emile G. Perrot.*

Designers of reinforced concrete structures cannot overestimate the advantage of experiments on full size members, such as columns and beams. Before the United States Government and the universities had undertaken to clear the clouded atmosphere concerning the design of reinforced concrete, one had to depend on the experience gained from actual structures and the test loads put on these buildings. As the number of buildings erected was limited, the available data was, of course, meager; but with the immense volume of work constructed to-day, a study of the behavior of various types of floor construction would appear to be both useful and instructive.

The analysis of the results of test load on buildings will deal with work that has come directly under the author's immediate notice, and embraces buildings erected during a period of about 8 years. These tests were made, not with a view of obtaining scientific data, but more particularly of satisfying both the architects and the owners that the work of the contractor had been properly performed and that the building would sustain the loads for which it was designed. Hence criticism of the somewhat crude method of determining the deflections should be suspended.

Regarding the so-called arch action of the load, the author was never able to notice such a phenomenon. Various kinds of materials were used to get the desired weight, according to their availability, but in every case a certain unit of the material was weighed on scales in advance to determine how much load was placed on the floor.

The deflection in most instances was measured by means of a transit, targets first being set up on top of the floor and the readings taken before loading. During the loading of the floor, a close watch was kept of the deflection so that in case any defect

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had occurred in the work to weaken the construction, it would be noticed before sufficient load was placed to create a failure. In addition to transit readings another method used to find the deflection was by means of two strips of wood or rods, one suspended from the ceiling of the floor being tested, the other supported on the floor below. Where the two rods overlapped a mark was made, before loading, and the deflection measured from this mark.

In the earlier tests, the amount of the floor area tested and the overload were smaller than at present. The practice now is to require a floor to be tested to double the live load without sign of fracture and that after the load is removed the floor must recover its normal position. It is the writer's belief that many specifications require a too rigid test by imposing the requirement of loading the floor to three or more times the live load. A little consideration will show the fallacy of this requirement, since it is not desirable to test an actual floor of a building so as to stress the reinforcement to a point equal to or greater than its elastic limit, as this would permanently weaken the section of the floor so tested. Take for example a floor designed for a live load of 200 lbs. per sq. ft.; assume the test load to be three times the live load; also assume the dead weight of the construction to be 75 lbs. per sq. ft. Then as usually computed with a factor of safety of 4, the breaking load of the floor would be 1,100 lbs. per sq. ft. If from this is deducted the dead weight of 75 lbs. per sq. ft., the live load to break the floor is 1,025 lbs. If a test load of three times 200 lbs., or 600 lbs. per sq. ft. is applied, there is likelihood of the reinforcement being stretched beyond its yield point, or elastic limit, since the average elastic limit of medium steel is one-half its ultimate strength. Hence a test load that exceeds more than one-half of 1,025 lbs., or about 500 lbs., should not be applied. This, it will be noticed, is about 2½ times the live load. The requirement of the Bureau of Building Inspection of Philadelphia, for a test load, is 2 times the live load.

Observations of the behavior of the various floors tested has convinced the writer that with the use of high elastic limit steel the deflections are considerably less than when mild steel is used, providing the unit stresses allowed in the calculations and

TABLE I.—TEST LOADS ON REINFORCED CONCRETE FLOORS.

Building.	Member.	Clear Span, ft.—ins.	Size, ins.	Reinforcement.	Net Deflection. ins.	Span Center to Center. ft.—ins.	Ratio of Depth to Span.	Test Load, lbs. per sq. ft.	Calculated Live Load, lbs. per sq. in.	Kind of Reinforcement.	Remarks.
Thos. Leedom and Sons. First Floor.	Girder	.....	None	.....	.....	.....	.....	200	75	Square twisted.	
	Beam	24—3	8x15	3—1¼″ sq.	1/16	25—0	1/18				
	Slab	7—4	5	5/16″ @ 5″ c.	.....	8—0	1/19				
Victor Talking Machine Co. Power House. Second Floor.	Girder	.....	Steel	.....	3/64	.....	.....	Total load 240,650	Total load 73,700	Medium.	
	Beam	18—0	10x18	4 sq. ins.	.....	19—4	1/8				
	Slab	1—2	12	.....	.....	2—0	.....				
Wm. Montgomery and Co. Fourth Floor.	Girder	13—8	11x23	8—1″ rd.	1/16	15—2	1/8	250	250	Medium.	Panel frozen.
	Beam	19—3	11x20	4—1 3/16″ rd.	1/16	19—9	1/8				
	Slab	7—0	5½	3/8″ sq. @ 6″ c.	3/8	7—7	1/16				
Friedenwald Company. First Floor.	Girder	17—4	11x24	8—1 1/16″ rd.	3/16	19—0	1/8	460	250	Medium.	
	Beam	14—6	8x14	4—7/8″ rd.	.....	15—1½	1/10				
	Slab	4—1	4	3/8″ rd. @ 6″ c.	.....	4—9	1/14				
Friedenwald Company. Second Floor.	Girder	17—4	11x24	8—1 1/16″ rd.	3/16	19—0	1/8	500	250	Medium.	
	Beam	14—6	8x14	4—7/8″ rd.	.....	15—1½	1/10				
	Slab	4—1	4	3/8″ rd. @ 6″ c.	.....	4—9	1/14				
Friedenwald Company. Third Floor.	Girder	17—4	11x24	8—1 1/16″ rd.	3/16	19—0	1/8	545	250	Medium.	
	Beam	14—6	8x14	4—7/8″ rd.	.....	15—1½	1/10				
	Slab	4—1	4	3/8″ rd. @ 6″ c.	.....	4—9	1/14				
Buck Building. Third Floor.	Girder	15—6	12x24	6—1″ sq.	1/16	17—2	1/7	234	200	Square twisted.	
	Beam	18—0	8x18	2—1″ sq. 1—1¼″ sq.	1/16	18—10½	1/10				
	Slab	5—2	5	3/8″ sq. @ 6″ c.	3/32	5—9	1/14				
Buck Building. Fifth Floor.	Girder	15—6	12x24	6—1″ sq.	.....	17—2	1/7	308	200	Square twisted.	
	Beam	18—0	8x18	2—1″ sq. 1—1¼″ sq.	1/16	18—10½	1/10				
	Slab	5—2	5	3/8″ sq. @ 6″ c.	.....	5—9	1/14				
Buck Building. Sixth Floor.	Girder	15—6	12x24	6—1″ sq.	.....	17—2	2/7	320	200	Square twisted.	
	Beam	18—0	8x18	2—1″ sq. 1—1¼″ sq.	1/16	18—10½	1/10				
	Slab	5—2	5	3/8″ sq. @ 6″ c.	.....	5—9	1/14				

Sheip and Company. First Floor.	Girder	16—6	12x22	8—1¼″ sq.	1/16	18—6	1/4	450	300	Square twisted.	
	Beam	16—0	8x18	4—1″ sq.	1/8	16—11	5/8				
	Slab	5—6	5	½″ sq. @ 4″ c.	.....	6—2	1/15				
Boyertown Building. Ninth Floor.	Girder	15—0	12x24	5—1″ s l.	.....	16—11	1/2	240	120	Square twisted.	
	Beam	24—9	8x21	2—1⅛″ 1—1″	none	25—9	1/13				
	Slab	5—0	3½	5/16″ sq. @ 6″ c.	.....	5—8	1/19				
Victor Talking Machine Co. Machine Shop. Third Floor.	Girder	20—6	12x26	6—1⅛″ sq.	none	22—4	1/8	300	150	Square twisted.	
	Beam	15—0	8x12	2—1″ sq. 1—¼″ sq.	3/16	16—9	1/14				
	Slab	4—11	4	.37 sq. in. @ ft.	.....	5—7	1/14				
School of the Most Precious Blood. First Floor.	Girder	.....	Steel	.....	.....	.....	.....	140	120	Medium.	Frames medium. Rods square twisted.
	Beam	28—9	8x18	2—1 5/16″ sq. 2—1″ sq.	1/2	29—3	1/16				
	Slab	4—4	3½	5/16″ sq. @ 9″ c.	.....	5—0	1/12				
Goodman, Loeb and Co. First Floor.	Girder	21—6	12x24	7—1″ sq.	1/32	23—3	1/10	244	120	Square twisted.	
	Beam	15—0	6x12	1—1″ sq. 2—¾″ sq.	3/32	16—0	1/16				
	Slab	5—4	4	5/16″ sq. @ 6″ c.	.....	5—0	1/15				
Warren Webster and Co. Second Floor.	Girder	15—3	12x26	6—1″ rd.	1/16	16—4	1/7	220	170	High carbon.	Corrugated bars. Deflection of short span 1/16 in.
	Beam	25—2	5x15	1—¾″ sq. 1—⅞″ sq.	1/8	26—2	1/18				
	Slab	2—7	2½	¼″ rd. @ 6″ c.	——	3—0	.....				
Academy of the Immaculate Heart. First Floor.	Girder	.....	.....	.....	.....	.....	.....	100	50	Square twisted.	Concrete joist and block filler.
	Beam	20—0	4x12	1 3/8″ sq.	1/16	21—0	1/17				
	Slab	1—0	2	¼″ rd. @ 3 ft.	.....	.....	.....				
Club House, Philadelphia Turngemeinde. First Floor.	Girder	53—0	16x36	21—1¼″ sq.	¼	55—3	1/13	240	120	Square twisted.	Concrete joist and block filler.
	Beam	12—5	5x12	1—1 1/16″ sq.	.....	13—9	1/12				
	Slab	1—0	2	.....	.....	.....	.....				
Victor Talking Machine Co. Cabinet Factory. Second Floor.	Girder	23—6	12x24	5—1 1/16″ sq.	13/64	25—0	1/10	333	150	Square twisted.	
	Beam	14—0	8x10	3—1″ sq.	¼	15—0	1/12				
	Slab	12x14—0	6	½″ sq. 6″ c. both ways	⅛	.....	1/30				
Victor Talking Machine Co. Cabinet Factory. Second Floor.	Girder	23—6	12x24	5—1 1/16″ sq.	3/16	25—0	1/10	364	150	Square twisted.	
	Beam	13—4	8x10	3—1″ sq.	3/32	14—4	1/12				
	Slab	12x13—4	6	½″ sq. 6″ c. both ways	3/32	.....	1/30				

ratio of depth of beam to span are the same. Hence the overloading of a floor where high elastic limit steel is used may be perfectly safe, where with mild steel reinforcement the same overload might cause excessive deflection and crack the concrete on the tension side of the beam. One case came to the writer's notice where this very thing happened. While the floor was not seriously damaged, there remained the objection of having permanent cracks in the bottom of a beam. This, of course, is a strong point in favor of the use of high carbon or twisted steel. While some authorities do not permit the use of higher unit stresses when figuring concrete reinforced with high elastic limit steel, the logic of the situation is not quite clear, since, to take an analogous case, one would not be justified in limiting the unit stress on yellow pine to that used for white pine.

It was the writer's intention to so plot the results of the tests that an approximate formula could be deduced, which would give the deflection of a beam for various spans and loading, but the information at hand was not complete or uniform enough to accomplish this much desired end. Experience has shown, however, that fine cracks appear in a beam where a deflection occurs, of from 3/16 to ¼ in. in a span of about 20 ft. Usually a floor loaded to twice the live load will not deflect any more if the ratio of over-all depth of beam to span is not less than 1/12; for slabs, however, a much smaller ratio seems perfectly safe. When the slab is reinforced in one direction only and has continuous action, a ratio of 1/20 is perfectly safe, while for a slab reinforced in two directions a ratio of 1/30 is permissible. This is the ratio of the slab in the Cabinet Factory of the Victor Talking Machine Company, and under test deflected 3/64 to ⅛ in. on a span of 15 ft.

In uniformly loaded test beams with free ends, the writer has seen cracks develop near the center of the beams, which were 20 ft. long, where the total deflection was about ¼ in. Whether invisible cracks appeared before is not known. Also in actual buildings fine cracks have occurred where one day's work joined another in the middle of the span, when a deflection of 13/64 in., or a little over 3/16 in. occurred in a span of 23 ft. 6 ins.

Table I shows the effect of test loads on floor panels of various designs. Special attention is called to the stiffness of the

floor of the Boyertown Building, where no appreciable deflection was noticed on a span of about 26 ft., under a load of twice the live load; also to the slight deflection of the concrete joist floor of the Academy of the Immaculate Heart, under a test of double the live load. The total deflection was only 1/16 in. on a clear span of 20 ft. with a ratio of depth of floor of 1 to 17.

The long T-beams which support the gymnasium floor of the Philadelphia Turngemeinde exhibit remarkable stiffness, the total deflection amounting to only ¼ in. in a span of 55 ft. under a load of twice the calculated load. This is a deflection of only 1/2540 of the span.

In all the above cases the reinforcement used was cold twisted bars. In contrast to these it is worthy to note the excessive deflection of the floor of the school of the Most Precious Blood, where the greatest deflection was ½ in. in a span of 30 ft. under a load of not quite 1½ times the calculated live load, and this with depth of floor of about 1/11 of the span. The reinforcement in the latter case is plain medium steel.

All of the above floors, except that of the first building mentioned, were constructed by the same reinforced concrete contractor, who is experienced in this work, so that the character of the workmanship was about the same.

These facts lead one to the conclusion that floors reinforced with high elastic limit steel are far superior, in carrying capacity, to those reinforced with medium or mild steel, all other conditions being the same.

THE WEB REINFORCEMENT OF CONCRETE BEAMS.

By John Stephen Sewell.*

Ten years ago the first step in the discussion of this subject would probably have consisted of a demonstration of the necessity or desirability of using web reinforcement for concrete beams. At the present time most engineers who have studied the subject agree that, under conditions of average practice, web reinforcement is at least highly desirable, if not absolutely essential to safety. There remains a considerable diversity of opinion as to the amount and kind of reinforcement that should be used. A purely theoretical study of the subject, if carried to any refinement, is quite complicated, and not amenable to any general mathematical discussion. Just as the question of crippling stresses in the web of a steel girder is one that must be considered for each individual case of loading—especially where heavy concentrated loads occur—so the web reinforcement of concrete beams presents many diverse problems. The practical study of the subject is also complicated, owing to the fact that every test is a special case, and the results are not generally applicable without modification. Fortunately a large number of tests is now available, however, and they cover the ground sufficiently well to disclose the fundamental necessity for web reinforcement, and also afford a guide to reasonably safe practice, even under conditions not exactly paralleled in any one test or group of tests.

In February, 1906, the writer presented a paper† in which certain views on web reinforcements were set forth in some detail, including methods of design and calculation. The tests that have been made since then have not caused the writer to change his views in any essential particular, but they have thrown new light on a good many points that were then obscure. Without endeavoring to discuss these tests mathematically, the writer desires to emphasize some points of general interest brought out by them, and to indicate what seems to him their practical application.

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†See *Transactions*, Am. Soc. C. E., Vol. LVI, pp. 252-288.

Before taking up the tests, however, the writer desires to present some examples which will show at once why web reinforcement is a matter of practical importance. There is still at least one engineer who maintains that web reinforcement, by stirrups or special web members, is unnecessary. He has worked out a system of design which, if followed, will undoubtedly give safe results—at least as long as the concrete is not cracked by expansion and contraction at any vital point and provided that the structure is loaded as assumed in the design. He works out a case, as follows:

1. Given a floor with a live load capacity of 100 lbs. per sq. ft. uniformly distributed. The floor slab is supported by beams, spaced 5 ft., center to center, with a span of 20 ft. There results a 4-in. slab, with suitable reinforcement, the design of which is open to no serious objection. The beams are 20 ins. deep and 15 ins. wide, reinforced with 3.75 sq. in. of steel. Assuming reinforcement to cost 2.5 cts. per lb. in place, the cost of this beam lin. ft. would be about as follows:

Reinforcement, 12.75 lbs. at 2.5 cts.	$ .319
Concrete, 2.1 cu. ft. at 20 cts.	.420
Centering, per lin. ft.	.300
Total cost per lin. ft.	$1.039
Weight of beam per lin. ft.	303 lbs.
Weight of beam and slab per sq. ft. of floor	93 lbs.

Concrete is assumed to weigh 140 lbs. per cu. ft.

2. A beam for the same slab and live load, with the same spacing, designed in accordance with the methods outlined in the writer's paper above mentioned, would be 8 ins. wide, 21 ins. deep, with total reinforcement, including rigidly attached diagonal web members, of 8.75 lbs. per ft. The cost per lin. ft. would be as follows:

Reinforcement, 8.75 lbs. at 3 cts.	$ .265
Concrete, 1.17 cu. ft. at 20 cts.	.234
Centering per lin. ft.	.290
Total cost per lin. ft.	$ .787
Weight of beam per lin. ft.	170 lbs.
Weight of beam and slab per sq. ft. of floor	79 lbs.

3. It will be interesting to compare the above concrete beams with an equivalent steel beam construction. For the conditions stated, 12-in. I beams 31.5 lbs. per ft., spaced 6 ft. 8 ins. center to center, with 12-in flat terra cotta arches would be suitable. The cost of a beam and its skewbacks per lin. ft.—together with some allowances for concrete fill over the top flange—would be about as follows:

Steel, 31.5 lbs. at 3 cts.	$ .945
Skewbacks, etc.	.350
Total cost per lin. ft.	$1.295
Weight of beam per lin. ft.	31.5 lbs.
Weight of beam and terra cotta per sq. ft. of floor	50.0 lbs.

The cost of the three designs per sq. ft. of floor would compare about as follows:

1. Concrete beam and slab, no web reinforcement	$ .49
2. Concrete beam and slab, with web reinforcement	.45
3. Steel beam and terra cotta arch	.50

By taking advantage of continuity and some other latitude in design, which would be permissible with the use of attached web members, the second reinforced concrete beam could be made somewhat less expensive and considerably lighter. It would then show a greater economy as compared with the steel beam, fireproofed; a more marked saving would be effected in the case of the girders supporting the beams. These points will be referred to later.

There are also certain structural considerations that enter here, in the case of the three beams in question.

Should it be desired to concentrate heavy loads near the ends of the beams, if the supports are adequate, as they might be, where only an occasional beam was to be so loaded, the only restriction in the case of the steel beam would be the crippling of the web, and in the case in hand, a total load of 17,800 lbs. might be distributed near each end, provided its center of gravity did not fall more than 2.5 ft. from the end. In the case of the second concrete beam, it would be safe to so concentrate one-half of the total load for which the beam was designed, i. e. about 9,000 lbs. at

each end. In the case of the first concrete beam, it is doubtful whether such a concentrated load could safely exceed about 5,000 lbs. at each end.

It is manifest that under conditions very likely to arise in storage warehouses, for instance, the first concrete beam might easily be subjected to dangerous web stresses; the writer has seen a number of such cases where the fact was evidenced by diagonal cracks. The second concrete beam would not be overstressed as to the web, except in very rare cases. The steel beam, from this point of view, is the safest of all. When the writer was in the Government service, he was frequently called upon to determine how much load could be concentrated at the ends of the beams. He always specified one-half of the total safe distributed load, concentrated within the end quarters of the span. This rule would almost always be safe for the second concrete beam, but not for the first one.

It appears to the writer that this question of end concentration on reinforced concrete beams is a matter of much practical importance, and that it has not yet received the attention it deserves.

In designing the web reinforcement of concrete beams a uniform load is nearly always the condition assumed—in building work, at any rate. The tests made with loads concentrated at the third points show very plainly that more web reinforcement ought to be used near the center of the span than would be required under uniform loading, if heavy concentrated loads are likely to occur.

Taking up the question of actual tests now available the well standardized ones made at the University of Illinois and the University of Wisconsin are very fruitful. Familiarity with these tests is assumed.

Vertical Stirrups.

One deduction often made from these tests is that vertical stirrups are of no value until after the concrete has cracked, and then only useful to insure ultimate collapse primarily by tension in the steel. It appears to the writer that this conclusion is not entirely borne out by the tests themselves, although it may be correct within limitations.

Referring to *Bulletin No. 29, University of Illinois,* in Table 9 are given the results of the tests of a large number of beams without web reinforcement. In Table 14 are given the results of a considerable number of tests of beams with vertical stirrups.

It happens that certain groups of beams in the two series are absolutely alike in every respect, except the presence of stirrups. The following groups have been selected as examples.

Group 1. Beams without Web Reinforcement.

Beams Nos. 251.1, 251.2, 251.3, 251.4.
Concrete, 1, 2, 4. Age at test, 60 days.
Span, 6 ft.
Reinforcement, 4½ in. corrugated bars.
Percentage of reinforcement, 1.25.

All failed by diagonal tension.

Average load at first diagonal crack ..... 15,000 lbs.
Average maximum applied load ..... 15,125 lbs.

Group 2. Beams with Vertical Stirrups.

Beams Nos. 232.1 and 232.2.
Span, 6 ft.

All data same as for four above mentioned, including brand of cement, except that Nos. 232.1 and 232.2 had each 6 vertical stirrups in the outer third of the span at each end; spacing of stirrups 4 ins.

Average load at first diagonal crack ..... 26,000 lbs.
Average maximum applied load ..... 39,150 lbs.

These beams failed by tension in the reinforcement.

Group 3. Beams without Web Reinforcement.

Beams Nos. 252.5, 252.7, 253.6.
The same as beams 251.1 to 251.4, except that another brand of cement was used, and the beams here considered were a few days older.

Average load at first diagonal crack ..... 22,000 lbs.
Average maximum applied load ..... 24,730 lbs.

They all failed by diagonal tension.

Group 4. Beams with Vertical Stirrups.

Beams Nos. 232.5 and 232.6.

Same as above, but with 6 stirrups in each end, as in 232.1 and 232.2.

Average load at first diagonal crack ..... 18,000 lbs.
Average maximum applied load ..... 23,500 lbs.

These beams failed by diagonal tension.

If the first two groups alone are considered, it would appear that there is a very great advantage in the stirrups, and that they must have begun to take diagonal tension from the start. Exactly the opposite conclusion would be drawn from the last two groups. But both indicate that the stirrups are the cause of a greater increase in load from the first crack to failure, which is at least a desirable feature.

All of the tests indicate wide differences in the quality of concrete as between corresponding groups of beams, even when the same cement was used, and the mixture was the same.

It is possible to select other groups of beams from the same tables; a few of them show, apparently, considerable advantage from the stirrups; more of them show no advantage in the use of stirrups. On the whole, these two series of tests appear to justify the negative conclusions stated in the bulletin as to the value and influence of stirrups. The writer, however, is inclined to think that there is in these tests a balance of positive evidence in favor of them, but that variations in the quality of the concrete largely masked the real influence of the stirrups. The tests also give evidence of the value of close spacing for the stirrups; this is emphasized by comparison with the results reported in *Bulletin No. 14, University of Illinois,* where the spacing of the stirrups exceeded the depth of the beam, and they were clearly of little value.

In *Bulletin No. 12, University of Illinois,* are reported the results of some remarkable tests with "T" beams, reinforced with plain and corrugated bars, and vertical stirrups. Here there were 5 stirrups of ½ in. corrugated bars in each end of each beam, spaced 6 ins. center to center, just outside of the load points. The span was 10 ft. and the beams were loaded at the third points. The beams were 11 ins. deep, over all.

These beams were all heavily reinforced, some with plain round bars, some with corrugated bars. All failed by tension in the reinforcement. The percentage of reinforcement was quite large, varying from .92 to 1.10 per cent. of the enclosing rectangle, which was equivalent, in some cases to about 4 per cent. of the stem alone, from the bottom of the beam to the top of the flange. The stress developed in the longitudinal reinforcement at failure was as high as 64,000 lbs. per sq. in., and the web stresses developed were almost unbelievable. It is stated that "the metallic web reinforcement was efficient and adequate." In some of the beams, a part of the longitudinal bars was bent up, but in others, just as heavily reinforced, they were all straight. Those with the bars bent up seemed to derive some advantage from that fact, but all of them gave results better than the writer would have believed possible. He thinks the conclusion as to the efficiency of web reinforcement is a very conservative one, indeed. While these beams had the advantage of a rather long span, they were, in some cases, so crowded with reinforcement that their ability to develop its elastic limit is one of the things that would hardly be believed, but for the fact that they did it.

Again, in *Bulletin No. 28, University of Illinois,* the tests of 3 large reinforced concrete beams there described certainly warrant the conclusion that there is a very real value in vertical stirrups, and that they must, in some cases, take tensile web stresses from the start—just as they must have done in the "T" beams.

The tests reported in *Bulletins Nos. 175 and 197, University of Wisconsin,* bear out, in a general way, the conclusions to be derived from the similar tests at the University of Illinois. The Wisconsin tests seem to indicate that vertical stirrups begin to take tension only after diagonal cracks have formed; yet they contain some tests on "T" beams from which exactly opposite conclusions could be drawn, just as in the case of the "T" beams tested at the University of Illinois.

The bulletins from both Universities point out certain peculiarities in the action of vertical stirrups, and the appearance of diagonal cracks. In many cases, vertical stirrups, especially in the vicinity of the load points, show a preliminary compressive deformation, followed by extension, after the diagonal cracks have

formed. The diagonal cracks often begin to open in the middle of the beam, not reaching the bottom until they are very well defined in the lower half of the concrete above the reinforcement; this is especially the case where no web reinforcement is provided. These points are very interesting and the writer has sought for some rational explanation. The following suggestions, based on a very general theoretical consideration, are offered.

It is known that a homogeneous solid beam, under transverse stress, is transversed by lines of principal stress, both tensile and compressive.

In Fig. 1 the right hand half indicates the general arrangement of these lines of principal stress in a homogeneous beam under uniform load. The full lines c are lines of compression, and the broken lines t are lines of tension. As is well known, the

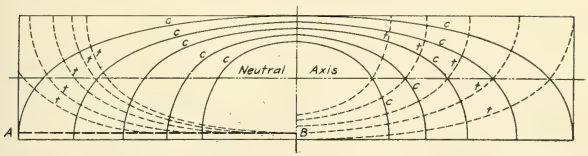

FIG. 1.—GENERAL ARRANGEMENT OF LINES OF STRESS IN A HOMOGENEOUS BEAM UNDER UNIFORM LOAD.

summation of the horizontal components of the stresses acting in directions tangential to lines c and t, gives the flange stresses for which beams are designed.

Now, in a reinforced concrete beam, under uniform loading, we may conceive that most of the lines of tensile stress pass through the horizontal reinforcement, and emanate from it, somewhat in the manner indicated in the left hand of Fig. 1, by the broken lines t. The compression lines c are shown as distributed in the same way as in the right half; this is not known to be correct, but they would probably not suffer as much modification as the tensile lines.

At any vertical section of the right half of the figure, the sum of the vertical components of the compressive stresses will be equal and opposed to the vertical components of the tensile stresses. A vertical member inserted here, imbedded in the beam,

would be equally affected by these opposing stresses, and would, therefore, not be stressed itself. The case, of course, would be different, if, at the section considered, a heavy concentrated load were applied to the top of the beam.

Considering the left half of the figure, it might easily be possible to place a vertical member where the sum of the vertical components of the compressive stresses would be in excess of the sum of the vertical components of the tensile stresses; and whether the diagram is even approximately correct, or not, the tests seem to indicate that this is not only possible, but that it actually happens. If the diagram be accepted as correct, it is also evidently possible to find sections where the vertical tensile stresses will exceed the vertical compressive stresses, and the tests also bear out this assumption.

If, instead of uniform loading, concentrated loads are applied to the top of the beam, either of the above conditions may be considerably aggravated. In the one case, the vertical stirrups will undoubtedly be in compression at first and will not come into tension until diagonal cracks have formed; in the other, they will be in tension from the start.

To determine the exact distribution and intensity of the web stresses in a reinforced concrete beam by any theoretical analysis appears so difficult and uncertain that the writer is not disposed to attempt it. To determine these things by experiment is equally difficult, to all appearances, and, at any rate, it has not been done. No attempt will be made therefore to apply the above reasoning to any individual tests; it is merely offered as a possible explanation of certain apparently inconsistent results shown by the tests. The practical conclusion, in the writer's mind, is, that while under certain conditions of loading, vertical stirrups do take tension from the start and accomplish their intended purpose, for beams liable to great variations in the distribution of the load, as most beams in practice are, vertical stirrups are, at the best, of doubtful value, and often useless. As better means are available, it seems best not to depend upon vertical stirrups, whether attached to the main bars, or not.

A consideration of the left half of Fig. 1 also suggests an explanation of the fact that diagonal tension cracks make their

appearance below the neutral axis and above the reinforcement. Above the neutral axis, the tensile stress becomes more nearly vertical, and less intense. Near the reinforcement it is more intense, but also so nearly horizontal that the horizontal reinforcement takes up most of it, and prevents the cracks from appearing there until after they appear above, and until the deformation of the horizontal reinforcement is sufficient to crack the concrete. The so-called tension cracks that open in the bottom of the beam may thus be due, in some measure, to web stresses. They generally show an inclination in a diagonal direction as soon as they extend above the reinforcement—which tends to confirm this hypothesis.

Diagonal Reinforcement.

The distribution of the principal stresses in a solid beam indicates very clearly that tensile web reinforcement will act to the best advantage if it is diagonally arranged, so as to be parallel, in a general way with the average direction of the tensile web stresses. From a consideration of the direction of the principal stresses in a homogeneous beam, it has been deduced that the best angle for diagonal web members is one of 45 deg. But a study of the actual tests, combined with the fact, that, from a theoretical point of view, the presence of the horizontal reinforcement undoubtedly tends to disturb the distribution of the tensile web stresses in a manner indicated in the left half of Fig. 1, would indicate that a direction making a smaller angle—probably somewhere between 35 and 40 deg.—with the horizontal, would be better. As the concrete in the web of the beam is the locus of both compressive and tensile stresses, and as an imbedded reinforcing member is bound to be affected by any stress existing in the concrete having a parallel component, it is necessary to consider the compressive stresses in connection with the web reinforcement. The writer is inclined to think that this point has not hitherto been clearly brought out in discussions of this subject. If the web reinforcement meets the compressive stresses at right angles, the net result is merely an increase in the bond between it and the concrete. The angle that would accomplish this, if it could be accurately determined, would manifestly be

the proper one. But as it would not be the same for all conditions of loading, it cannot be accurately determined. If the web members are more nearly vertical than this angle would make them, in any case, they will surely take some compressive stress from the concrete, and their efficiency will be impaired; if they are more nearly horizontal, they will suffer an increase in tensile stress by virtue of the compressive web stresses.

The latter alternative, not carried too far, is manifestly the one to be adopted. As above indicated the writer is inclined to believe that, as a general proposition, the proper angle probably lies between 35 and 40 deg. with the horizontal.

Not only theory, but all the recorded tests as well, indicate that diagonal web members, so arranged, will surely take tensile stress from the beginning, and that they will be just as efficient in preventing the formation of diagonal cracks as the main reinforcement is in preventing the formation of ordinary tension cracks. In fact, under the assumption of a rectangular stress strain function, they should be more efficient, if properly designed, for the reason that the stress will be transmitted into them, above the neutral axis, at a uniform rate, whereas the rate of change of the stress in the main reinforcement is an increasing one, as the abutments are approached, when the loading is uniform, or approximately so. This results in a much greater intensity of bond stress for the main bars near the ends than in the middle of the span. Were the conditions reversed, so that the greater bond stress came in the center where the tensile stresses in the concrete are also most severe, it is probable that the early experiments would have given such discouraging results that reinforced concrete would have been abandoned as impracticable, although, in the light of more recent experience, this would not be a justifiable conclusion.

Diagonal web reinforcement may be secured by bending up all or a part of the main bars, at various points of the span, or by the use of separate diagonal web members.

Bent Up Bars.

The value of diagonal reinforcement of this kind has been demonstrated quite conclusively by numerous tests. It was

clearly pointed out in certain patents, and demonstrated by many tests of the system in Europe. Among American experiments, those by Mr. J. J. Harding* were very striking. The tests reported in bulletins from the University of Illinois and of Wisconsin, above referred to, are likewise corroborative.

These tests speak for themselves, and will not be discussed in detail, but certain general conclusions may be drawn from them.

1. *It is not good practice to bend up all the bars.* Certain beams in which this was done, had the bars anchored at the ends, with nuts and washers. (See *Bulletin No. 29, University of Illinois,* Beams 514.1, 514.2, 514.3, 541.1 and 541.2.) These beams developed high web stresses, and all but one failed by tension in the reinforcement. The loads, however, were applied almost exactly over the bends in the bars. The loading was the one that the beams were best adapted to stand. Had the loads been applied at the two quarter points—or even a little closer to the supports—and had they been combined with a load at the center, it seems probable that the results would have been less satisfactory. In the case of the beams with all the bars bent up and not anchored at the ends, reported in the same bulletin, although the loading was the most favorable possible, the failures were all by slipping of the bars at the ends or by diagonal tension. A different distribution of the load would probably have aggravated the undesirable features of these failures.

2. *Bars should be bent up at various parts of the span,* and should be sufficiently numerous to distribute the web reinforcement through all parts of the beam subjected to serious web stresses.

This is indicated in the tests reported in *Bulletin No. 197, University of Wisconsin,* and is strongly emphasized in the tests of three large reinforced concrete beams, reported in *Bulletin No. 28, University of Illinois.*

In the tests reported in Bulletin No. 197, many of the beams had bars bent up at two points in each half of the span, and, in some cases the angle was less than 45 deg. The bent up bars were also carried along the top to the ends of the span, giving good

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anchorage. Some of these beams developed very high web stresses. While they all contained numerous vertical stirrups, which seem to have been reasonably effective, it is hard to avoid the conclusion that the bent up bars were a more important factor in the results.

In the case of the three large beams reported in Bulletin No. 28, Beam 70 had bars bent up very near the ends; for some distance outside of the load points, there was no web reinforcement. This beam failed by diagonal tension between the load point and the first bent up bars. The other beams had bars bent up almost under the load points, and also had some vertical stirrups. Only one of them was tested to failure; it failed by tension in the reinforcement. In this beam the angle in the first bent up bar was also less than 45 deg.

All three of these beams developed high load carrying capacity under the conditions of the tests. Yet, in view of the marked difference in behavior due to variations in the relative positions of the loads and the web reinforcement, the writer is not sure that it is justifiable to expect these beams to be as satisfactory under conditions of service, which involve rolling loads in the shape of locomotives and trains, as they were under the test conditions. It is assumed that they were designed for very conservative working stresses, so as to avoid cracks in the concrete. But if railroad bridge loadings should go on increasing in the future, as they have in the past, these beams will probably be quite severely tried.

In *Bulletin No. 29, University of Illinois,* are reported tests on four beams, Nos. 521.1, 521.2, 521.5, and 521.6, reinforced with a proprietary system, in which the principle of bent up bars is applied in an admirable manner. These beams failed by tension in the reinforcement, and developed exceptionally high web stresses.

However, in all of the reported tests above referred to, in which the bars were bent up, there were developed diagonal cracks which indicate that the web reinforcement was not wholly adequate, and that different loadings might have caused even the best of these beams to fail by diagonal tension.

Diagonal Stirrups.

There are not many reports of accurately conducted tests on beams in which reinforcement is provided by diagonal stirrups, as distinguished from bars bent up.

In *Bulletin No. 197, University of Wisconsin,* however, are records of several such tests. The stirrups were, in some cases, loosely attached, in others rigidly attached, to the main bars. The conclusions, thoroughly justified by the tests, are, that inclined stirrups are superior to vertical stirrups of the same kind, and that stirrups rigidly attached to the main bars are superior to loose stirrups. After remarking that the percentage of main reinforcement used was not great enough to thoroughly test the web reinforcement, the experimenter makes the following significant statement: "How efficacious a reinforcement, which concentrates a large portion of its metal in single web members, would be in resisting high diagonal tensile stresses is a matter for further experiments to determine."

Whether the above question has since been investigated, the writer does not at present know. But it is easy to find in the recorded tests available, abundant grounds for the experimenter's opinion.

There are still but few reported tests on beams reinforced with diagonal web members attached to the main bars. By courtesy of Professor A. N. Talbot of the University of Illinois, the writer is able to submit the record of some tests made on beams reinforced with a patented reinforcement in which the principle of diagonal web members, firmly attached to the main bars, is very neatly and effectively applied.

The reinforcement in the case of these beams was assembled as it would be for uniform loading. The writer is strongly of the opinion that even the fine diagonal cracks that appeared could have been almost prevented by a different arrangement of the web members. Nevertheless, the beams failed in an ideal manner; they carried remarkably large loads, considering the rather low elastic limit of the reinforcement, and they developed very high web stresses.

The company controlling the patents on the form of rein-

TABLE I.—RESULTS OF TESTS OF CONCRETE BEAMS REINFORCED WITH SPECIAL BARS.

Concrete mixture, 1 2, 1–3, 5. Size of beam, 8 x 11 ins. Depth of reinforcement, 10 ins.

Two special units of mild steel. Total area of longitudinal steel, 1.28 sq. in. Reinforcement, 1.6 per cent. Four web members at each end of each bar. Each web member consisted of a loop of $\frac{1}{4}$-in. round wire, making in all four sections of wire at each point of bending up. The length of beams, 78 ins. Loaded at one-third points. All stresses are given in lbs. per sq. in. Span, 72 ins.

	Cubes.		Control Beams.		Reinforced Beams.							
						Load.						
Beam No.	Age, days.	Ultimate Strength, lbs.	Age, days.	Modulus of Rupture.	Age, days.	Yield Point.	Maximum Load, lbs.	Bending Moment, lb.-ins. at Maximum Load.	Shearing Stress.	Bond Stress.	Load at First Crack, lbs.	Deflection in Inches Yield Point of Beam.
282.1	103	3,223	77	375	64	29,000	32,000	389,000	240	970	14,000	0.12
282.2	95	4,758	70	430	69	30,000	32,000	389,000	240	970	10,000	0.15
282.3	86	3,210	73	340	65	30,500	33,850	411,000	260	1,030	9,000	0.15

NOTE.—Failure occurred by tension in the longitudinal reinforcement. Tests were continued beyond this point, and with the continued stretch of the longitudinal bars load increased somewhat, deflections increased considerably, and crushing of the concrete at the top of the beams was finally apparent before or at the maximum load, though, of course, this may not be called the cause of failure.

forcement used in these beams informed the writer that Professor Talbot was incorrectly informed as to the exact weight of the bars; that they were ¾ in. instead of 8/10 in. bars. If this is correct, then the percentage of reinforcement was only 1.4 instead of 1.6. The stress in the reinforcement under maximum load, if the bars were ¾ in., averaged about 42,500 lbs. per sq. in. If they were 8/10 in., about 37,600 lbs. The record of the tests is submitted in Table I, just as it was received from Professor Talbot.

Notes on Tests Scheduled in Table I.

No. 282.1. At a load of 14,000 lbs. and a deflection of 0.428 ins.

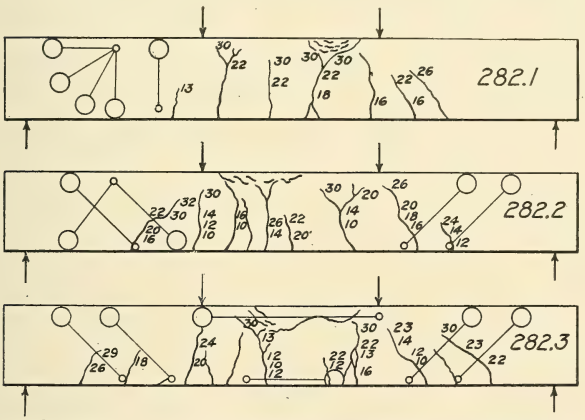

FIG. 2.—DIAGRAM SHOWING LOCATION OF CRACKS AND INSTRUMENTS.

the first crack appeared under left load. (See Fig. 2.) The second and third cracks appeared at a load of 16,000 lbs., one being a diagonal crack, 6 ins. to the right of the right load. At 22,000 lbs. the first crack forked at the mid-height of the beam. Crack No. 6, which appeared at the 18,000 lb. load, also forked in the same manner. At 29,000 lbs., initial failure occurred, apparently by the reinforcement near the center of the beam reaching the yield point. At 32,000 lbs. the concrete failed by crushing at the top of the beam as shown in Fig. 3.

No. 282.2. At a load of 10,000 lbs. and a deflection of .0348 in. the first cracks were noted; one 9 ins. to the right of the center extended 5 ins. high and was somewhat inclined, one 8 ins. to the left of the

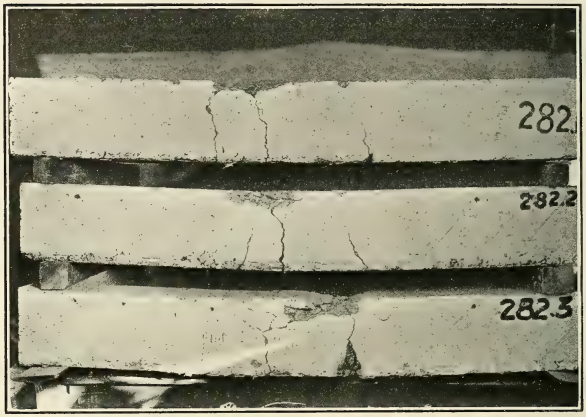

FIG. 3.—TYPICAL BEAMS AT FAILURE. NOTE THE HOLES DRILLED FOR EXTENSOMETERS AND COMPARE FIG. 2.

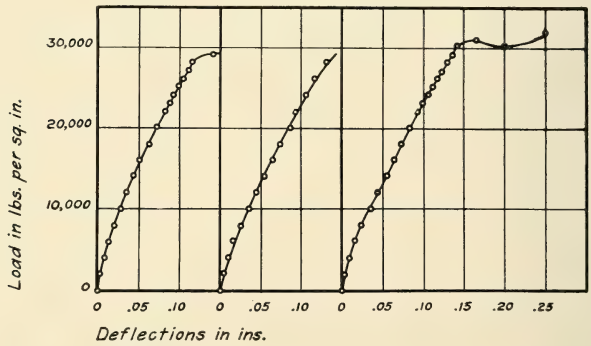

FIG. 4.—STRESS-STRAIN CURVES FOR BEAMS 282.1, 282.2 AND 282.3.

center was 4 ins. high, and one 14 ins. to the left of the center was 3½ ins. high. At 16,000 lbs. the two principal diagonal cracks appeared. At 26,000 lbs. the diagonal cracks on the right had extended to within 3 ins. of the load point. At 30,000 lbs. the cracks near the center of the beam were opening rapidly and one forked like those of beam No. 282.1. These cracks opened to within 2 ins. of the top of the beam, and at 32,000 lbs. the concrete crushed as shown in Fig. 3.

No. 282.3. The first cracks developed in this beam at 3 ins. and 11 ins. left of the center at a load of 9,000 lbs., the one nearest the center showing only above the reinforcing bar and not to the bottom of the beam. At 10,000 lbs. a diagonal crack was noted above the first web member on the right. At 18,000 lbs. a diagonal crack from junction of web member on the right developed. This crack never crossed the first web member. At 22,000 and 25,000 lbs. cracks developed at the junctions of the third web members right and left. At 30,000 lbs. the crack near the center began opening rapidly and at 32,900 lbs. the concrete began to crush at the top, the load remaining constant until the deflection reached 1 in. The load then rose slowly, reaching the maximum at 33,850 lbs., with a deflection of 1.2 ins. At the load of 32,500 lbs. and a deflection of 1.4 ins., a large piece of concrete split up between load point, as shown in Fig. 3. A large deformation occurred in the web members at the right end of the beam.

Stress-strain curves for the three beams are given in Fig. 4.

It will be observed that in these tests there was a combination of very strong concrete, with probably a high modulus of elasticity, and a reinforcement of rather soft steel. As Professor Talbot points out, this combination is likely to develop visible cracks in a rather early stage, even when the maximum load ultimately required to produce complete failure is very high.

The percentage of reinforcement was not as high as could have been desired. There should have been at least two more web members on each half of each main bar; the spacing should have been uniform, or nearly so, and the inner web members should have had their points of attachment well inside of the load points. It would have been better, also, to have made the inner web members longer, and have set them at a flatter angle. The writer is inclined to believe that had this been done, the owners of this bar would have had a very striking result. As it is, the results of the tests are a little better than any others at all comparable with them, but not as much so as those commercially interested would probably have been glad to see.

The writer has been convinced for a number of years, that the ideal reinforcement for concrete beams consisted of main bars, with many small web members rigidly attached, and set as nearly as possible parallel with the average direction of the tensile web stresses. There have been very few tests accurately made, on beams with such reinforcement. The tests reported in *Bulletin No. 197, University of Wisconsin,* and these three on special bars, at the University of Illinois, are the only ones the writer knows of, that even approximate the conditions, and they fall considerably short of the full requirement.

A number of practical tests, consisting of the application of load in the form of pig iron, iron balls, etc., have been made, which bring out the value of the system in a very forcible manner. But the records are scattered and inaccessible, and not as complete as they should be, so no attempt is made here to collect them. Most of them were made with a view of closing contracts, and it might be urged against them that they are not to be accepted at their face value. One of them, however, was made by the French Government, at the request of the owners of the patents. Commercial bars were used; two beams were made identical in every respect except that one had diagonal stirrups riveted to the main bars, and the other had "U" shaped vertical stirrups, passing under the main bars. The beams were loaded with iron balls. The one with diagonal web members carried about 20 per cent. more load than the other, and failed gradually by tension in the reinforcement. The other failed suddenly, by diagonal tension, if the writer's memory serves him correctly.

It is much to be desired that this theory of attached web members diagonally placed, should be thoroughly tested out by disinterested experimenters like Professors Talbot and Withey, with reinforcement *built up from commercial shapes,* so as to be free of the taint of commercial interest. If the principle involved is as important as the writer believes it to be, the demonstration of its correctness should not be left to the self-interest of companies exploiting a patented method of applying it.

While the tests specially devised to try the principle out are very few and inadequate, it appears to the writer that among the phenomena attending the tests that have been made with other

systems of reinforcement, are many that plainly point to attached diagonal web members as the ultimate ideal. A consideration of the distribution of stresses in a reinforced concrete beam—and the modifications thereof necessarily produced by variations in the method of loading—would seem to lead to attached diagonal web members, without the evidence of tests or experience. A discussion of some of the details would seem to be in order at this point.

1. *Diagonal Arrangement.*—The reasons why this is preferable on theoretical grounds, have been set forth above in this paper. All the tests that bear on this point, without exception, confirm the theoretical conclusion.

2. *Rigid Attachment.*—The stresses in the main reinforcement are the horizontal components of the web stresses, both tensile and compressive. The fact that web reinforcement is necessary indicates that concrete is unreliable under the web stresses; if so, it is also unreliable as a medium for transmitting the stress from a web member into the main bar—i. e., for binding them together. All the tests that bear on this point confirm the above conclusion. But even if the tensile or shearing strength of concrete could be depended upon for isolated beams, as in laboratory tests, it would not follow that they would be so in practice, when contraction cracks, variation in quality of concrete, irregularity of mixing, etc., may easily destroy these properties in large measure, at the very points where they are most required.

3. *Multiplicity of Members, Attached to the Same Main Bar.*—Bending up the main bars at various points results in diagonal web members rigidly attached. But no main bar receives its stress at one point; the diagonal tensile stress is greater than its longitudinal component; only the latter goes into the main bar, as an increment of stress; but something must resist the entire diagonal stress. If the web members are set at an angle of 45 deg., the stress in the member itself is to its horizontal component as $\sqrt{2}$ to 1. It is evident, therefore, that even if matters could be so ordered that each main bar would receive the stress from only a certain portion of the span, and it were bent up so as to traverse the corresponding part of the beam, the bent up part must be more seriously stressed than the horizontal portion, in the proportion of $\sqrt{2}$ to 1. This makes its bond resistance a

matter of serious consideration, and also causes greater unit deformation at exactly the point where the deformation should be kept as low as possible. In fact, if the main reinforcement is to work under a stress of say 15,000 lbs. per sq. in., it is probable that the web members should be designed for not more than 12,000 lbs.

Again, in a long beam, it is a matter of practical impossibility to use so many main bars that one would be available for bending up at every point where a web member is required. As some of them must run through, in any event, the impossibility of obtaining the complete solution by bending up the bars alone is even more strongly emphasized.

The complete solution requires that the spacing, number and length of the web members for a given bar can all be varied. The web members should reach entirely to the top of the beam; they should be of such section that the part above the neutral axis, by its bond stress, can develop the full stress in the web member; they should be so numerous and closely spaced, that every section along which a diagonal crack might develop shall be transversed by enough web members to effectively prevent the formation of cracks between them under working loads. Of course, in tests to destruction, as failure approaches, the web members will stretch and permit diagonal cracks, just as tension cracks appear in the bottom of the beam, from stretching of the main reinforcement. But it is an easy matter to design the web members so that the stress in the main bars will pass the elastic limit before that in the web member reaches it.

The attachment between main bars and web members should be as strong as the web members themselves; it should permit of no lost motion in taking stress, and it should permit of their being bent upwards in gentle curves at first, starting from tangency with the main bars.

Reverting to the spacing, it is claimed that the presence of web members cannot relieve the concrete between them of web stresses. This is true, but it is equally true that the presence of the main bars cannot relieve the concrete around them and between them and the neutral axis of tensile stress. But if the web members are numerous enough, they can prevent the formation of tension cracks, until their own deformation is so great that the concrete can no longer stretch with them, just as the main bars do.

It is also argued that the web members, to be effective, should have an anchorage at their ends sufficient to develop their full strength at the centroid of compressive stresses in the concrete. But to the writer it appears very clear that the point where the full strength is first required is at the neutral axis. As the horizontal components of the stresses in the web members are equal, in the aggregate, to a certain percentage of the tensile stress in the main bars, so are they equal to the same percentage of the compressive stress in the concrete; and if the horizontal component of the safe unit bond stress above the neutral axis is equal to the rate of change in the compressive stress at every point, the web members will surely do their duty. Above the neutral axis, they no longer depend upon ordinary bond, but they are securely anchored by the compressive stresses in the concrete itself. To say that they must develop their full strength at the centroid of compressive stresses is exactly equivalent to saying that the main bars must develop their full strength at points, say 15 per cent. of the half-span from either support.

To fulfil all the requirements of a completely satisfactory and perfectly adaptable system of diagonal web members, rigidly attached to the main bars, is not easy. With commercial shapes, it involves considerable expense. It is, therefore, a fruitful field for patents. The writer knows of at least four patented systems, which apply the principle more or less perfectly. Three of them are on the market; but even among the patented systems, one very important problem is completely unsolved, and that is a means of attaching the same system of web members to both the direct and reverse reinforcement, in continuous beams.

The writer remembers distinctly, when in the Government service, his instinctive aversion to committing himself to the use of patented articles. Professional men generally seem to be actuated by the same feeling. It arises from motives that are entirely commendable, and it is better to carry it to the extreme of "standing so straight as to lean over backwards," than to commit their clients' interests to the mercy of commercial monopoly. The writer cannot wholly divest himself of this feeling, notwithstanding the fact that he is now engaged in commercial enterprises and is even slightly interested in two or three patents himself. But

he cannot avoid the conclusion that if any ready means of applying the principle of attached web members without infringing patent were at hand, the principle itself would by this time have met with general acceptance, and if the patented systems accomplished the same ends more economically, they would all be in a more flourishing condition commercially than they are. The engineers of a reinforced concrete bridge recently erected in Pittsburg seem to have found themselves in a predicament on this question, and to have escaped from it in a very creditable manner.*

While commercial questions are not entirely relevant in this paper, it seems to the writer that it would be a great boon to the reinforced concrete industry if the large steel manufacturing industries would acquire the various patents and sell the assembled reinforcement themselves, just as they do fabricated structural steel. From reliable data in the writer's possession, he is satisfied that they could sell the assembled reinforcement at a price that would make it cheaper in place than commercial bars with loose stirrups, and yet pay a handsome return on the additional investment. Of course, the average inventor's estimate of the value of his invention might prove a stumbling block, but it would be worth trying.

It may be argued that very satisfactory results are obtainable by the use of plain bars; that is perfectly true, but one reason for it is that, in this country, we design for loads that are rarely realized in practice There have been a good many cases where the loads assumed in the design have been equaled or exceeded in practice, and the results have not been satisfactory; still more where indifferent workmanship, which we have always with us, has prevented the concrete from acquiring the tensile and shearing strength requisite for satisfactory results. Nevertheless, when steel is very dear, and concrete cheap, reinforcement with plain bars can be used so as to make incombustible construction possible in many cases where the cost of steel frame construction would be prohibitive; and the writer would not condemn the use of plain material, if no better plan were available. It is better even to use cinder concrete for structural members

*See *Engineering News*, December 1, 1910, p. 583.

in many cases, than to use the flimsy combustible construction that is the only alternative.

But, in important structures, all considerations, both of economy and safety, are better subserved by attached web members.

In the writer's paper, above referred to, before the American Society of Civil Engineers, he showed that if reinforced concrete beams are designed by a certain formula, that their total cost is a minimum when the cost of the reinforcement is equal to that of the concrete above it. This principle applies, closely enough for all practical purposes, whether the formula referred to is used, or whether the design is made by more elaborate formulas, in which the variation in the position of the neutral axis is taken into account. But this principle calls for a percentage of reinforcement much too large for safety, if attached web members are not used. If they are used, this percentage is usually well within safe limits. The resulting economy does not stop with the cost of the beams themselves, but it lightens up the dead load in beams, girders, columns and foundations, and saves time in construction, because of the smaller quantity of material required.

The great certainty and economy of placing the reinforcement in assembled units is another great advantage. In long and heavy beams or girders, with attached web members, it is entirely safe and proper to stop off some of the main bars short of the supports, thus effecting another notable saving.

If continuity is accepted as inevitable, and the design made accordingly, the result is much more satisfactory with attached web members; and the saving in weight alone is a justification for the use of continuous beams and girders, not to mention the increase in rigidity of the structure as a whole. But in many important structures the question runs to continued safety, especially in buildings subject to heavy vibrations and repeated loads, or in railroad structures.

To leave such work dependent for its safety upon the continued integrity of the bond of the main bars, is, in the writer's judgment, risky at the best. This involves depending upon the shearing strength of concrete under conditions that are far from ideal.

The paper on the Fatigue of Concrete,* by Professor J. L.

*Transactions, Am. Soc. C. E., Vol. LVIII, June, 1907.

Van Ornum, shows that the bond will undoubtedly be seriously diminished by the repeated application of loads far short of those sufficient to produce failure with one application. Some tests at the University of Illinois point plainly in the same direction. The tests reported in *Bulletin No. 321, University of Wisconsin,* are apparently more favorable, but it appears to the writer that the method adopted in this case amounted merely to plain pull-out tests in another form, and the results are not applicable to beams.

The study of bond friction resistance in reinforced concrete,* by William Fry Scott, points again very plainly to the risk of depending upon the continued integrity of the bond The same conclusion follows from a study of the paper on Investigations on the Slips of Rods Imbedded in Concrete,† by H. Burchartz.

Where reinforcement consisting of plain bars, whether part of them are bent up or not, or of bars with loose stirrups, is used, the ultimate safety is absolutely dependent upon the continued integrity of the bond. In structures that are subject to overloads, or to the continued repetition of the working load, the writer thinks there is sufficient ground for anxiety as to the future, with most of the ordinary systems of reinforcement. How long it will be before the trouble appears, is perhaps a matter for speculation. But the writer has seen it in its incipient stages more than once. What may ultimately happen to some of the railroad viaducts and bridges that have been built in recent years, he thinks is a matter for very serious consideration. It ought to be settled by extensive laboratory tests before it has time to develop in practice.

With properly designed attached web members, the continued integrity of bond around the main bars is probably of small importance, and there can be little doubt as to the bond around the web members themselves, in the compressed part of the concrete. But, if under the repetition of severe loads, even this should ultimately fail, there will be warning enough to prevent a catastrophe.

Of the advantages of attached web members in a fire, the

Proceedings, Am. Soc. S. E., October, 1910.

†*Engineering Record,* November 12, 1910.

writer has expressed his views at length in the paper above referred to, and in other places, and will not here repeat them. But that a reinforced concrete structure may come through a fire apparently intact, and still be seriously damaged in reality, was apparent enough from a study of the ruins of San Francisco, and was quite clearly indicated by the example in *Engineering Record* of November 12, 1910, p. 535, in the comment on Resistance of Concrete to Fire. With attached web members the concrete can be appreciably damaged on the exposed lower surfaces and for some distance upwards, without seriously affecting the structural safety. As ordinarily built, reinforced concrete buildings offer the possibility of suffering a loss that is "more than total," if such a thing be possible; in other words, of yielding a negative salvage, after the fire, because they may be so badly damaged as to require rebuilding, yet cost a great deal to destroy. Attached web members would make adequate repairs possible in many such cases, thus avoiding the expense of wrecking the building.

The writer believes that railroads will find the use of attached web members much more numerous than have ever been used, whether loose or attached, absolutely essential to the ultimate safety of structures subject to the passage of fast and heavy trains; and that many types of building will have the same experience. It is fortunate just now that we are prodigal in the use of materials; but the time is coming when we will have to figure more closely, and then many systems of reinforcement now in common use must disappear.

It appears from the tests on T beams at the Universities of Illinois and Wisconsin that there is a virtue in this type of member as regards web stresses not yet fully understood. It is to be hoped that further experiments will be made and studied until this feature is fully elucidated.

As for the general system of attached web members, the writer thinks it can be applied now, with obvious economy and much additional safety. But if the conflict between opposing commercial interests, based on various patented systems, is to delay the general acceptance and adoption of a principle that seems so vital to safety, then it would be a blessing if all the patents could be declared invalid and everything disclosed by them made public property.

DISCUSSION.

Mr. Berry. MR. H. C. BERRY (By letter).—The exhaustive study of the web stresses in reinforced concrete beams by Mr. Sewell, combining as it does a complete theoretical analysis and a careful examination of the published data from tests bearing on the question, has appeared at an opportune time to exercise a good influence towards establishing more conservative practice in design in this particular. There has been some concern among a good many engineers about hairline cracks developing at the ends of beams in a number of buildings throughout the East. A study of the load concentrations as hinted at in the paper in connection with a careful computation of the stresses possible from temperature may account in great part for these conditions. It is also probable that stresses due to shrinkage in hardening have a good deal to do with the formation of such cracks.

The general conclusions of the author are well founded, (*a*) that web reinforcement is very important, (*b*) that vertical reinforcement is effective, (*c*) that theoretical analysis and tests agree to indicate that inclined reinforcement is more effective than vertical, (*d*) that rigidly attached stirrups are better than loose ones, (*e*) that the inclination of stirrups should be between 30 and 35 deg. instead of 45 deg. as is usual. The author says that some of the conclusions and deductions are in need of further demonstration, especially if quantitative comparisons are desired. He deplored the scarcity of data on the actual measurement of web deformations. It was with the purpose to present some advance general conclusions from a very few tests in which the prime purpose was the measurement of the web deformations in beams with different forms of web reinforcement that this discussion is offered. The conclusions and general results are tentative and based on an insufficient number of tests to be final, but plots of the readings taken are so consistent among themselves that it seems desirable to offer them at this time.

The tests were made on three beams, one with no web rein-

forcement, one with vertical stirrups, and one with diagonal stirrups. The first and second beams were 7 ins. wide, 17 ins. deep with two ¾-in. rods for tension reinforcement, straight from end to end. The beam with vertical stirrups had ⅜-in. rods bent U shape and set 8 ins. between centers. The beam with diagonal stirrups was 13 ins. deep with ⅜-in. rods bent U shape and spaced 8 ins. between centers. Mr. Berry.

Short pieces of steel rod were cemented into the side of the beams and small holes 0.05 in. diam. drilled in these rods at 8 ins. between centers. As the loads were applied at the middle of the beams, readings were taken between these holes with a special strain gauge and the differences noted. The readings are reasonably accurate to the nearest 0.0001 in. in the 8-in. gauge length. This is equivalent to a stress of 375 lbs. per sq. in. in reinforcement or about 30 lbs. per sq. in. in the concrete on the usual assumptions as to the modulus of elasticity.

Because of the different conditions on the side of the beam it was thought best in making a critical study of the deformations to divide the side elevation of the half span into four parts. Above the neutral axis the shearing stress is variable while below, if we disregard tension in the concrete, the shear is uniform in any particular section. At the end of the span the bending stresses are minimum while at the middle they reach a maximum. It seems very probable that the actual deformation is considerably affected by the two kinds of stress at any particular place. For instance, it seems reasonable to assume that the lower part of the beam at the ends of the span would show deformations least affected by the presence of bending stresses, while measurements in the top of the beam near the point of application of the load are affected by the compression horizontally due to the bending and the local vertical compression from the load itself.

The plain beam developed 102 lbs. per sq. in. average vertical shear and failed in diagonal tension; the beam with vertical stirrups developed 80 lbs. per sq. in. average vertical shear and failed in bond; the diagonal stirrups beam developed 107 lbs. per sq. in. average vertical shear and failed in the tension reinforcement. Readings of the deformations were taken at intervals up to a load causing 75 lbs. per sq. in. vertical shear for the plain, 55 for the

Mr. Berry. vertical, and 90 for the beam with diagonal stirrups. These readings were taken between points vertically above each other, between points at 45 deg. from the horizontal above and below looking from the middle toward the end of the beam. Some readings were taken on horizontal gauge lengths in the upper and lower parts of the beams.

Unfortunately the load increments were not taken so as to give exactly similar vertical shearing stress at high values for direct comparison. At about 50 lbs. per sq. in. vertical shearing stress the differences in amount of the deformation for the different beams were very small. The most striking thing is the small amount of this deformation in all of them, especially in the vertical direction.

Reduced to equivalent stress in reinforcement for the measured deformation the vertical measurements for the beam with no web reinforcement show a stress of from 750 to 2,200 lbs. per sq. in.; with vertical reinforcement, from 650 to 2,700 lbs. per sq. in.; the diagonal web reinforcement beam, 600 to 3,600 lbs. per sq. in. This would indicate that at that load, causing about 50 lbs. per sq. in. vertical shear, there is little stress in the contained reinforcement.

In the case of the beam with diagonal stirrups a set of observations was made at a load causing an average vertical shearing stress of 90 lbs. per sq. in.

In the direction of the stirrups the deformations were equivalent to that which would occur in steel at stresses from 5,500 to 9,800 lbs. per sq. in. The average of 6 different gauge lengths was 7,700 lbs. per sq. in.

In the vertical direction the deformations were equivalent to a stress of 11,800 lbs. per sq. in. in one case and 12,600 in another. One gauge length was inconsistent.

In a direction across the diagonal stirrups the compressive deformations were equivalent to that caused by a stress of 4,000 lbs. per sq. in. in steel.

At these small deformations hairline cracks might be present but they would be very fine if visible at all; careful examination showed none.

The tests made so far do not furnish sufficient data to show

that for working loads a smaller inclination of diagonal stirrups is to be sought and it is not shown that at 50 lbs. per sq. in. vertical shear that diagonal stirrups are more effective than vertical ones, but it is apparent that for either arrangement the working stress to be assumed in the design of the web reinforcing should be taken very low. Mr. Berry.

Mr. Maurice Goldenberg (By letter).—Mr. Sewell has very clearly and ably reviewed the facts relating to the use of web members and emphasized the logical deduction, from such tests as exist, that the use of web members in general is of advantage and that the most efficient type is inclined and rigidly attached to the main tension members. Mr. Goldenberg.

It is incomprehensible to the writer, in view of such indisputable facts, why inclined rigidly attached web members and their attending advantages are not more generally demanded by engineers. Mr. Sewell infers one possible reason, namely—that such a type of reinforcement is patented and controlled by private interests who demand unreasonable prices. This statement might possibly have been based upon conditions as they existed many years ago, but they are not applicable to-day. Patented bars of the efficient type, advocated by Mr. Sewell, can be purchased at the base price of ordinary plain bars, plus a reasonable charge for shop work or fabrication. This latter amount in fact is actually less than the ordinary cost of supplying and placing loose stirrups. Therefore, this superior type of reinforcement with all its advantages, may be used at a less first cost than plain bars. There is not the slightest doubt about this statement, as it is founded on a very extensive use of rigidly attached inclined web reinforcement.

In view of this fact, Mr. Sewell's attitude toward manufacturing companies, controlling patented articles, is unjustifiable. On the contrary, such companies deserve the thanks of the engineering profession for enlisting the skill and experience of the best procurable engineers to further the development of reinforced concrete. Thousands of dollars are spent annually by such companies in research work tending toward more efficient and economical materials as well as their application. It would be quite serious if manufacturing concerns did not keep pace with the progress of the theoretical engineers, and it is just as reason-

Mr. Goldenberg.

able that they should receive a profit as the engineers should receive monetary compensation. It is very creditable to the existing reinforcement manufacturers that their business is so capably managed that they can afford to do such research work, supply such improved materials, keep pace with the advances of the best engineers and, at the same time, sell their products at such reasonable prices.

Possibly another reason why web members have not been more generally used is the tendency to adhere to the use of such materials or methods as have been previously used with apparently good results. It is wrong to assume that, because no poor results appeared as the result of the use of loose stirrups as web reinforcement, that that type is the best. It is reasonable to believe, in view of the tests and theory referred to by Mr. Sewell, that rigidly attached inclined web members, with other conditions being the same, would give better results.

In judging a finished structure, one must be exceedingly careful where he places the credit for apparently good results. Oftentimes, if the structure is analyzed with sufficient care, the real results may prove to be not as good as was believed. The ordinary structure, such as a building, is generally designed with a liberal factor of safety, based upon a low average of strength of materials. Its floors are usually (except in certain warehouses) designed for a safe carrying capacity greatly in excess of what is ordinarily required. There is always much variation and uncertainty in the quality of the concrete ingredients, mixing, placing and the general conditions surrounding the construction of particular structures, which conditions materially affect the results. Therefore, one can understand why there is sufficient variation, aside from the reinforcement, to counteract any possible deduction as to the special efficiency of the arrangement of the reinforcement itself.

The only certain way of determining the advantage of rigidly inclined diagonals over the ordinary loose stirrups, is to make comparative tests on beams prepared under the same conditions, identical in every respect, with the exception of the one feature that is to be tested, namely—the type and arrangement of the web members. Tests of this nature have been referred to by

Mr. Sewell, but the writer wishes to herewith submit a table of the results of these and other tests, so arranged as to afford ready comparison and easy deduction. Mr. Goldenberg.

The tests made at the University of Wisconsin are more fully described in Bulletin No. 197, University of Wisconsin, by Morton

TABLE I.—TESTS MADE AT UNIVERSITY OF WISCONSIN.

Beams paired together between blanks are to be compared together.

Beam No.	Width of Beam. Inches.	Effective Depth. Inches.	Span. Feet.	Reinforcement.	Maximum Load. Lbs.	Stress in Steel. Lbs. per sq. in.
N1	5	13.65	10	One 1-in. sq. bar, 10 $\frac{7}{8}$ x $\frac{1}{4}$-in. loose vertical stirrups	24,000	39,400
N2	5	13.65	10	One 1-in. sq. bar, with rigidly connected inclined shear members	*34,600*	*57,400*
N3	5	13.65	8	One 1-in. sq. bar, 12 $\frac{3}{8}$-in. loose vertical stirrups	26,000	34,300
N4	5	13.65	8	One 1-in. sq. bar, with rigidly connected inclined shear members	*37,400*	*49,600*
N5	8	11.75	10	Two $\frac{3}{4}$-in. sq. bars, with rigidly connected inclined shear members	*32,600*	*62,100*
N6	8	11.75	10	Two $\frac{3}{4}$-in. sq. bars, 14 $\frac{1}{4}$-in. with loose inclined stirrups	24,000	45,500
N7	8	11.75	10	Two $\frac{3}{4}$-in. sq. bars, 20 $\frac{1}{4}$-in. with loose vertical stirrups	22,000	38,800
N8	8	11.75	10	Two $\frac{3}{4}$-in. sq. bars, with rigidly connected vertical shear members	*28,000*	*43,700*
N9	8	11.75	8	Two $\frac{3}{4}$-in. sq. bars, with rigidly connected inclined shear members	*47,200*	*60,200*
N10	8	11.75	8	Two $\frac{3}{4}$-in. sq. bars, 12 $\frac{1}{4}$-in. with inclined loose stirrups	34,000	36,700
N11	8	11.75	8	Two $\frac{3}{4}$-in. sq. bars, with rigidly connected vertical shear members	*39,000*	*48,300*
N12	8	11.75	8	Two $\frac{3}{4}$-in. sq. bars, 16 $\frac{1}{4}$-in. with loose vertical stirrups	34,000	45,000

Tensile Tests on Steel:—
Elastic limit (average) = 39,650 lbs. per sq. in.
Ultimate strength (average) = 67,400 lbs. per sq. in.

Owen Withey, Instructor in Mechanics, under whose direction the tests were made. Tests made by the University of Missouri were conducted by students under the general direction of the Engineering Department. Tests made at the University of Michigan were a thesis prepared by Chas. E. Hart. In the test made at Paris, a flat bar was used for reinforcement, to which the rigidly connected web members were riveted. Further details of this very interesting test may be found in *Concrete and Constructional Engineering,* London, May, 1906.

Mr. Goldenberg.

TABLE II.—TESTS MADE AT UNIVERSITY OF MISSOURI.

Beams paired together between blanks are to be compared together.

Beam No.	Width of Beam. Inches.	Effective Depth. Inches.	Span, Feet.	Reinforcement.	Maximum Load. Lbs.	Stress in Steel at Initial Failure. Lbs. per sq. in.
M1	5	14.4	10	One 1-in. sq. bar, loose vertical stirrups	34,800	46,750
M2	5	15.4	10	One 1-in. sq. bar, with rigidly connected inclined shear members	*44,200	57,200
M3	5	13.7	8	One 1-in. sq. bar, loose vertical stirrups	43,300	46,000
M4	5	15.0	8	One 1-in. sq. bar, with rigidly connected inclined shear members	49,800	55,700
M5	8	14.2	10	Two $\frac{3}{4}$-in. sq. bars, with rigidly connected inclined shear members	43,000	58,500
M6	8	14.0	10	Two $\frac{3}{4}$-in. sq. bars, loose inclined stirrups	37,200	42,500
M7	8	13.7	10	Two $\frac{3}{4}$-in. sq. bars, loose vertical stirrups	34,300	48,550
M8	8	14.0	10	Two $\frac{3}{4}$-in. sq. bars, with rigidly connected vertical shear members	38,850	44,900
M9	8	14.0	8	Two $\frac{3}{4}$-in. sq. bars, loose inclined stirrups	47,100	47,100
M10	8	14.0	8	Two $\frac{3}{4}$-in. sq. bars, with rigidly connected inclined shear members	50,000	54,800
M11	8	14.0	8	Two $\frac{3}{4}$-in. sq. bars, with rigidly connected vertical shear members	46,600	55,700
M12	8	14.2	8	Two $\frac{3}{4}$-in. sq. bars, loose vertical stirrups	45,500	47,900

* In Beam M2, testing machine broke at this load.

Tensile strength of steel = 67,000 lbs. per sq. in.

TABLE III.—TESTS MADE AT UNIVERSITY OF MICHIGAN.

Beam No.	Width of Beam. Inches.	Effective Depth. Inches.	Span Feet.	Reinforcement.	Maximum Load. Lbs.
1	4.09	5.33	4	One $\frac{1}{2}$-in. sq., with rigidly attached diagonal web members	6,250
2	4.06	5.34	4	Twenty-one $\frac{1}{8}$-in. round rods and one hundred and twenty $\frac{1}{8}$-in. round rods, loose vertical web members	4,850

TABLE IV.—TEST WORK AT CONSERVATOIRE NATIONAL DES ARTS ET METIERS, PARIS.

Beam No.	Width of Beam. Inches.	Effective Depth. Inches.	Span. Feet.	Span. Inches.	Reinforcement.	Maximum Load. Lbs.
1	6	$13\frac{3}{4}$	16	$4\frac{7}{8}$	One $\frac{1}{2}$ x 2-in. bar, with eighteen loose vertical members	22,339
2	6	$13\frac{3}{4}$	16	$4\frac{7}{8}$	One $\frac{1}{2}$ x 2-in. bar, with eighteen rigidly attached inclined web members	26,830

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Referring to tests made at the University of Wisconsin, it will be noted that the beams reinforced with rigidly attached web members developed an average tensile stress of 80 per cent. of the ultimate strength of the reinforcement and carried a load 30 per cent. greater than those reinforced with loose web members, which latter only developed approximately 59 per cent. of the ultimate strength of the reinforcement.

Referring to the tests made at the University of Missouri, the beams reinforced with rigidly attached web members developed approximately 81 per cent. of the ultimate strength of the reinforcement and carried 12½ per cent. greater load than those reinforced with loose web members, which latter only developed a strength of 69 per cent. of the ultimate strength of the reinforcement.

It will be interesting in the University of Wisconsin tests, to compare beam N-5 with N-8 and N-9 with N-11, all of which four have rigidly attached web members, but two of which are vertical and two inclined. A comparison indicates that the inclined web members produced 19 per cent. greater strength than the vertical web members, other conditions being equal. A similar comparison may be made with loose web members to show that the inclined web members produce greater strength than the vertical, other things being equal. In the same way, one might analyze the tests made at the University of Missouri with similar results in favor of rigidly attached inclined web members.

In the French official comparative test, the rigidly attached inclined web members produced in the beam, a carrying capacity of about 21 per cent. in excess of the beam reinforced with loose vertical stirrups. After the test, these beams were carefully destroyed so as to expose, as far as possible, the real action of the reinforcement. It was found in the beam containing the loose vertical members, that the latter had slipped at each place where a crack in the beam occurred, thus substantiating the reasoning outlined above and proving that there must have been some force tending to slide the bar past the web members, which force was resisted in the case of rigid attachment.

Referring to the tests made at the University of Michigan, a single bar was compared with 141 bars, including loose web mem-

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bers with an ultimate strength of 28 per cent. increase in favor of the rigid attachment. In this test it will be noted that, at a load of 4,850 lbs., the deflection of the loose member beam was .145 in.; whereas, with the rigidly attached members, the deflection was .065. The deflection of the rigidly attached members at 6,000 lbs. was .25. This excessive deflection, at a given load, with loose stirrups indicated quite conclusively some slipping between the web members and the main bar.

It will also be interesting to note the method of failure in beams using these two classes of web reinforcement. It appears that the rigid attachment produces a gradual failure with much ultimate deflection and warning. The beams with loose web members generally fail with the failure of the concrete, thus being sudden and without a great deflection.

These tests indicate clearly that beams reinforced with rigidly attached inclined web members possess greater strength, have less deflection at a given load, greater deflection before failure and a gradual failure at ultimate loading, as compared with beams with loose vertical web members, all other conditions identical.

The action of web members has often been compared by engineers:

1. To a truss, in which the reinforcement resists all the tension stresses and the concrete all the compression.

2. To the dowels or pins connecting a series of horizontal plates, of which the beam, in theory, is composed.

If web members resemble in action, similar members in a truss, then surely the necessity for rigid attachment is obvious. If the member be inclined, then its horizontal component must be transmitted to the main tension member and sufficient connection between the two must be provided. If the member is vertical, then the compressive web strains are inclined and its corresponding horizontal component must be transmitted to the main tension reinforcement. This is accomplished either by depending entirely upon uncertain adhesion or by considering a rigidly attached web member as an abutment. Therefore, whether the web member be vertical or inclined, if the action resembles that of a truss, and this assumption looks very reasonable, then the web members should obviously be rigidly attached to the main member.

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If a reinforced concrete beam resembles a series of horizontal layers, bound together by reinforcement in such a way that all layers, including the reinforcement which is one of the layers, act together as a single unit, then it is even more obvious that the web members or dowels, whether they be inclined or vertical, must be rigidly attached to the layer of reinforcement; otherwise, they would not act together and the principle of a series of layers would be absurd. This principle is well illustrated by comparing with a flexible book which, when the leaves are loose, will bend laterally very readily by the sliding of one leaf past the other. However, if all the leaves are rigidly connected so that they cannot slide by each other, then considerable stiffness is imparted to the book. This stiffness depends entirely upon the thoroughness with which the various leaves are held together and the whole made to act as one. Loose stirrups bind together all the layers of concrete but permit the reinforcement to act independently, thus preventing a development of the greatest strength.

It may be interesting to consider what is the best practical section for web members in reinforced concrete beams. This feature is dependent upon the method of receiving stress. If tension exists in a web member, as undoubtedly it does, it can receive this tension only by adhesion or compression against the concrete. Many engineers have advocated that, in order to prevent the slipping of such web members, they should be well anchored in the top of the beam. However, the writer has never, either in practice or in tests, seen a single case of slipping of a web member even where, under ordinary conditions of adhesion, the length of the stirrup was not sufficiently great to receive the stress which undoubtedly it carried. The reason for this is brought out by Mr. Sewell in the following statement:

> If the web reinforcement meets the compressive stresses at right angles, the net result is merely an increase in the bond between it and the concrete.

and in another place:

> Above the neutral axis, they no longer depend upon ordinary bond, but they are securely anchored by the compressive stresses in the concrete itself.

Mr. Goldenberg.

It is reasonable, therefore, that if the compressive stress in the beams plays such an important part in assisting the web members in resisting pulling out, that the best section or shape of web members is flat, as such a section would give a wider area to meet the compressive stresses, and thus increase its bond.

This same flat section is also well adapted when the web member is intended to act as an abutment to resist the inclined compressive stresses and transmit its horizontal component into the reinforcement. In such cases, the stresses bear directly against the web member and are greatest in intensity at the junction of the web member and the main bar. Therefore, if at that point, the web member is flat, it will offer a wider area to resist the compression and prevent a crushing of the concrete at such points.

An interesting experiment was made on a beam composed of a reinforcing bar with rigidly attached diagonals, between which were loosely placed wood blocks. There was an entire absence of adhesion as the reinforcement was merely in contact with the wood. This beam, when tested, carried a surprisingly great load, which would have been much increased had it not been for the loose fitting joints and the compressibility of the wood. Further tests of a similar nature might develop some interesting facts regarding the supposed necessity of adhesion, particularly in the web members.

Mr. Sewell emphasizes deservedly the value of rigidly attached web members as an additional fire protection. A splendid example to illustrate this point is the building of the Dayton Motor Car Company, which suffered a severe fire in one story. Sufficient heat was developed to practically ruin the lower inch or so of concrete in the slab above. This destroyed all adhesion in the main tension members. However, upon testing, it was found that the strength of that floor was not diminished in the slightest. This was undoubtedly due to the rigidly attached diagonals which eliminated the necessity for adhesion in the main tension member. Undoubtedly, if loose stirrups had been used in place of rigidly attached members, the fire would have practically ruined the floor or, at any rate, greatly diminished its strength.

Mr. Sewell also emphasized the value of rigidly attached web members as a reinforcement where moving or vibrating loads are

to be resisted. This application is very important in factory buildings where the floors support heavy shafting, vibrating machines, suspended motors, etc., all of which combine to cause a rapid alternation and variation in stress tending toward the separation of the reinforcement and the concrete. Undoubtedly the life of a reinforced concrete factory building, subject to such vibrating loads, is greatly lessened if the strength depends upon adhesion. While the comparison is irrelevant, nevertheless one would have a more thorough appreciation of the seriousness of the effect of vibrating loads if he was acquainted with the condition of the ordinary mill constructed building after a few years of such service. Such buildings, even when constructed in the most approved manner, have to be substantially strengthened by the use of intermediate posts and other supports. The writer has known cases where, at the end of ten years, such buildings had to be torn down and rebuilt on account of insufficient strength to carry the loads for which they were originally designed. It is needless to say that, in a concrete building, if adhesion is depended upon, results of a similar nature might be expected under similar conditions. More than adhesion is necessary to indefinitely resist vibrating loads tending toward the separating of the reinforcement and the concrete. Mr. Goldenberg.

While it is true that many more tests are necessary to establish conclusive facts, nevertheless such tests as have already been made conform so closely to the theory and give such convincing results in favor of rigid inclined web members, as to warrant their further use, at least until sufficient more tests are made to establish absolute certainty. The writer believes that the results of all tests up to date point clearly to a general recognition and use of this type of web reinforcement in the very near future.

Mr. John Stephen Sewell (By letter).—The writer has read Mr. Goldenberg's discussion with great interest. Mr. Goldenberg evidently has misconstrued the writer's real attitude towards the owners of patented forms of reinforcement. The writer would be personally glad to see the owners of these patents secure a rich return for their ingenuity and investments. All that he intended to convey in his remarks on this subject was, that for the sake of real advancement in the art, considering the matter from the point Mr. Sewell.

Mr. Sewell. of view of the general good, the sacrifice of an individual might be justifiable.

With the other comments the writer is in practical accord, except that he does not think the flat form is necessarily the best for a web member. Carried to its ultimate conclusion, this might result in a thin sheet occupying an entire section of the beam. As a matter of fact, almost any ordinary form of bar is satisfactory for a web member, although it would be quite possible to make a very good theoretical argument in favor of the round bar as the best. The proper distribution of the web members along the axis of the beam is of more importance than the form of the members themselves. The writer thinks that Mr. Goldenberg might be able to find some examples where web members failed to do their duty because they did not extend far enough beyond the neutral axis of the beam. As he remembers it, a case in point is one of the first tests on this form of reinforcement made at the University of Illinois.

NOTES ON WEB REINFORCEMENT.

By Peter Gillespie.*

Considère believed that concrete in contact with steel under stress acquires properties entirely foreign to it when alone, and that under such circumstances, it possesses a capacity for extension without rupture, practically ten times as great as it exhibits when not influenced by the more elastic material. It has been one of the classic assumptions of theorists when discussing the mechanics of reinforced concrete, that the two materials deform together and partake, in a measure, of some of the peculiarities of a homogeneous element. The assumption that no initial stresses exist in either the reinforcement or the concrete is as old as the material concerning which it has been made. But theories are not finalities and are held only until such time as their weakness has been made known. The experiments of Moritz† quite satisfactorily prove that the first cracks in concrete under tension in reinforced concrete beams occur at or below a relative extension of .0001 to .0002, corresponding to a stress in the reinforcement of 3,000 to 6,000 lbs. per sq. in. The accompanying breaking tension in concretes of average quality will clearly not exceed 200 to 400 lbs. per sq. in. The experiments lead us to believe that concrete in conjunction with reinforcement is not radically different in extensibility from plain concrete. Moreover, the findings of Moritz were subsequently confirmed by Bach and others.

There recently appeared an abstract‡ of the results of some experiments made by Dr. Preuss, of the Technical College at Darmstadt, on the slip of rods imbedded in concrete. Martens' extensometers were employed in order to determine the movement, if any, of the reinforcement relative to the concrete in reinforced concrete beams under load. These experiments showed that a distinctly noticeable slip occurred even with very small loads, and

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†University of Wisconsin, *Engineering Series*, Vol. 3.

‡*Engineering Record*, Vol. 62, No. 20, p. 546.

that indeed before cracks were recognized. In order to discover readily the formation of cracks, the beams were coated with purified chalk. Determinations were made at the center, the quarter points and over supports, the loading being such as would give a constant bending moment over the middle half of the beams. A feature of the test was that at both middle and quarter points, the movement of the reinforcement relative to the concrete was rather capricious, being sometimes of one sense, sometimes of the other. Under small loads, a slip was sometimes observed in one direction which with larger loads, changed to the opposite direction. It was also noticed that with a given load, the movement might have one direction and after the release of the load, the reverse direction. In all cases where observations were made at the supports, the movement of the reinforcement relative to the concrete was toward the center. Generally speaking, conspicuous movement was observed to begin when the average bond stress developed between the two materials had reached about 80 lbs. per sq. in., this corresponding to a stress in the reinforcement in the middle half of the beam of 6,500 lbs. per sq. in. The experiments indicate that the movement increases rapidly as cracks develop in the vicinity of the region under investigation.

Recent investigations as to the shrinkage of Portland cement concrete lead us to believe that with improved methods of manufacture this peculiarity in the material may be almost completely overcome, that the stresses due to shrinkage which we now know are of considerable intensity, will be eliminated, and that, incidentally, the strength of concrete will be very much increased in consequence. As new light is shed, therefore, from time to time, we must be prepared to modify our views.

If the bond between the reinforcement and the concrete were perfect, an extension in the former would produce in the latter an infinite number of cracks, indefinitely close together but of infinitesimal aperture. There would be no localization of extension through the opening of larger cracks at finite intervals. If late investigations are reliable, and there seems no reason to doubt them, the reinforced concrete of commerce is not the material which some text books describe. The assumption that the concrete sustains no tension seems, however, to be justified. Unless the

stress in the reinforcement be limited to from 2,000 to 4,000 lbs. per sq. in. the material will rupture, throwing the entire load, or nearly the entire load, on the reinforcement. When a reinforced concrete beam is stressed, it would appear as though the rupture of the tension portion takes place long before the specified stresses in the reinforcement are realized. The extension is localized into perceptible or microscopic cracks, possibly separated by a few inches. (Moritz found them several inches apart in a beam of section 3 x 6 ins.) There will of necessity be local movement, with respect to the reinforcement, of the material between cracks, part of it possibly in one direction, part of it in the other. It will hence be recognized that methods of determining the position of the neutral axis and the stress in imbedded reinforcement by measurements made on the outside of a beam are likely to give erroneous results. An error of .16 mm. due to slipping (as found by Preuss in his tests accompanying a stress of 20,000 lbs. per sq. in.) might easily, in a gauged length of say 20 ins., occasion an error of 45 per cent. As confirmatory of this, Moritz found that for two beams identical in all respects except that the tension rods were exposed in the one and encased in concrete in the other, the deformation in the reinforcement for equivalent loads determined in the two ways, differed by 40 per cent. This movement will also explain why, even under quite heavy stressing, experiment has seemed to show that a considerable portion of the resisting moment in beams is sustained by the concrete in tension.

The attempt often made, to investigate the character and intensity of the internal stresses in a reinforced concrete beam with the aid of the theory of the homogeneous beam, is not entirely satisfactory, especially under moderate or heavy loading. It is quite common, for example, to picture the lines of maximum tensile stress in a homogeneous beam as constructed from the well-known formulas—

$$t = \tfrac{1}{2}f + \sqrt{\tfrac{1}{4}f^2 + v^2}$$

$$\text{and } \tan 2\theta = \frac{2v}{f}$$

where f represents the horizontal tensile stress, v the vertical or horizontal shearing stress, t the maximum tensile stress and θ the

angle made by the direction of t with the horizontal. In accordance with these equations, the lines of maximum tensile stress are horizontal at the bottom of the beam, since there v is zero, and intersect the neutral axis at an angle of 45 deg., since there f is zero.

If in a reinforced concrete beam, we can be sure that between the neutral plane and the metal, there is no horizontal tension, the concrete having ruptured under stress, it will follow from the mechanics of the beam, that the intensity of the shear in this same region will be constant. In the region above the neutral plane, the intensity of the shear horizontal and vertical, will follow the well known parabolic law, if the beam be of rectangular section. In accordance with the above equation, the maximum diagonal stress below the neutral plane will be equal to the shear, and its direction will be inclined upward at an angle of 45 deg. It is then commonly assumed that the concrete may be counted on to take a diagonal stress of say 40 lbs. per sq. in., and sufficient metal is supplied to carry the remainder, the spacing of stirrups of given section being arrived at by an adaptation of a well known rivet spacing formula for plate girder flanges. Now, in the first place, if concrete be incapable of sustaining horizontal tension, how is it expected to sustain inclined tension of equal or greater intensity? In the next place, if the concrete between the neutral plane and the reinforcement be divided into segments through rupture either vertically or obliquely, it will be manifest that the conditions existing in the homogeneous beam are not duplicated here, and the shear and diagonal tension equations cannot be considered as strictly applying. Under light stressing, perhaps, before it has produced rupture in tension, it would be permissible to employ them. But when the stress in the reinforcement has reached the commonly specified value of from 12,000 to 16,000 lbs. per sq. in., their use is exceedingly doubtful.

Assume a beam to sustain a uniformly distributed load, and assume also, that the load is an increasing one. There seems to be a justification for the belief that, at first, the two materials will deform together and there will be no rupture in tension. At the center of the beam, the lines of maximum tensile stress are all horizontal, and in consequence, subsequently, the rupture at this

region will be vertical. Toward the supports, the lines of maximum tension are inclined upward and outward, hence it follows that the lines of rupture will be inclined upward and inward. At the center as the stress in the reinforcement increases, the inevitable slip of rods added to the elastic deformation will cause the cracks to open, perhaps to a visible extent. Since the shear increases toward the supports, the obliquity of the ruptures increases also. Notwithstanding the presence of the vertical and inclined ruptures, the concrete between the neutral plane and the metal is still capable of resisting vertical and horizontal shear. More correctly, the concrete in the segments between such lines of rupture in the region referred to, is capable of resisting such stresses. A consideration of the fact that the tension portion of the beam is now probably divided into a series of segments separated by ruptures, will explain the observation of Dr. Preuss that sometimes the movement of reinforcement relative to concrete is of one sense, sometimes of another. The cracks must be regarded as points of accumulated movement, and as such it is perfectly comprehensible that movements of the metal relative to its encasing concrete may have opposite senses on opposite side of the center of the segment, let us say. The theory of the homogeneous beam cannot now be used to calculate the web stresses, for the portions thus separated must be considered rather as separate elements in a structure than as portions of a stressed material isolated by purely imaginary planes. Thus the theory is confined in its application entirely to initial conditions.

The theory which assumes that the vertical stirrups must carry the entire shear in a beam after the adjacent concrete has ruptured, is inadequate to explain the phenomena of beams under test, which is shown by an examination of the results of tests on beams M_1 and M_2, described in *Bulletin No. 197*, University of Wisconsin. These two beams were rectangular in section, 17 x 8 ins. and of 8 ft. span. They were reinforced with four ¾-in. deformed bars, and with twelve ¼-in. round stirrups each. They were equally loaded at two points each 1 ft. 4 ins. from center of span. In order to determine the behavior of the stirrups, two extensometers were attached to the first stirrup outside of each load point, one on each side of the beam, the gauge length being 12 ins. The readings on these extensometers were recorded and plotted and from them some

conception concerning the intensity of stress in these members may be had. On the assumption that the entire shear will be sustained by the vertical stirrups, the stresses in these members are also computed, and a comparison between the two results is interesting. It should be stated in explanation, that the extensometers on the stirrups were attached to the metal, not to the concrete outside of it. Manifestly, the stress over the 12 ins. of gauge length will not be constant, and so, any results based on that assumption must be in error; but, still will serve for a comparison. *A* and *B* refer respectively to the two prongs of the same stirrup on opposite sides of the beam. For beam M_1, the total elongations in 12 ins., sides *A* and *B*, respectively, were .0018 and .0027 ins., corresponding to stresses of 4,750 and 6,780 lbs. per sq. in. In beam M_2, the results were substantially the same. In Table XIV of this bulletin are given the stresses in these stirrups as 164,000 and 142,000 lbs. per sq. in., computed on the assumption that the entire shearing stresses are sustained by these members. Since it is stated that one of the stirrups shed its mill scale, it is safe to infer that it had been stressed past its yield point. Again, since the material in the stirrups is reported as having a yield point of 47,000 and an ultimate strength of 62,000 lbs. per sq. in., it is plain that neither value of the stress as computed can be correct. The results of the second method are too high and of the other too low, the latter possibly because the elongation as measured was not uniformly distributed over the length. That the stresses in the stirrup were first compressive need not occasion surprise. The load was applied in close proximity to the vertical stirrups on which measurements were made, and this doubtless explains the phenomenon. It is also apparent from the record that the metal in the stirrups is not stressed to any considerable extent until the adjacent concrete is cracked. It is evident that the web resistance supplied by the concrete after diagonal cracks have appeared, is due to that portion of the structure still in compression, and into which the diagonal cracks have not as yet penetrated.

In *Bulletin 28,* University of Illinois, are reported tests on three heavy concrete slabs 31 x 75 ins. x 25 ft. These were reinforced with 1.27 per cent. of deformed bars on the tension side and a small quantity of metal (about 0,04 per cent.) on the com-

pression side. They are designated as Nos. 70, 71 and 72. In No. 70, the web reinforcing material consisted of part of the longitudinal bars bent up toward the ends of the slab at an angle of approximately 45 deg. In Nos. 71 and 72, the same method was adopted except that the angles of inclination were in general smaller and that stirrups were supplied, spaced 18 ins. on centers in the direction of the length of the slab. These stirrups consisted of ½-in. deformed bars and constituted 0.1 per cent. of the volume of the slab. No. 70 failed by diagonal tension; No. 72 was a tension failure, the difference in kind being attributed to the difference in web reinforcement. The strength of the latter was some 5 per cent. greater than the former. There was, however,

TABLE I.—BEAMS WITHOUT WEB REINFORCEMENT.

Bulletin No. 29, University of Illinois.

Span 6 ft.; per cent. of metal, 1.25; age, 2 months; third point loading.

No.	Total Load at First Diagonal Crack, lbs.	Vertical Shearing Stress at First Diagonal Crack, lbs. per sq. in.	Stress in Longitudinal Reinforcement at First Diagonal Crack, lbs. per sq. in.	Maximum Applied Load, lbs.	Maximum Vertical Shearing Stress, lbs. per sq. in.	Maximum Stress in Longitudinal Reinforcement, lbs. per sq. in.
251.1	19,200	149	28,100	19,200	149	28,100
251.2	14,500	114	21,300	14,500	114	21,300
251.3	12,300	98	18,000	12,800	102	18,800
251.4	14,000	110	20,600	14,000	110	20,600
Averages	15,000	118	22,000	15,100	144	22,200

considerable evidence that the yield point of the metal in No. 70 had been well-nigh reached when failure took place suddenly, and so it required only a little additional strength in the web to change entirely the character of the failure. The maximum horizontal and vertical shearing stresses as computed by the usual formulas, were 209 and 220 lbs. per sq. in., respectively.

In order to determine the extent of the deformations in the stirrups, extensometers of span 28 ins. were attached to the outside of the beam as nearly opposite the known position of the vertical stirrups nearest to, but outside the load points. As observed on another occasion cited, the deformations appeared to be at first,

shortenings and afterwards extensions, the tension apparently coming into the verticals at about half-load, and then really only after rupture in the concrete had taken place. If the stress in these numbers be computed from the apparent deformations, it would come to 18,000 lbs. per sq. in. in one instance, and 25,700 lbs. per sq. in. in the other, while if it be assumed that the stirrups sustained the entire vertical shear, it would be something like 114,000 lbs. per sq. in. The case is quite analogous to the one referred to previously, and shows that the metal need not be counted on to sustain the entire shear in the web. Generally, unless it be approaching nearly to failure, the diagonal cracks do not invade seriously the com-

TABLE II.—BEAMS WITH BENT-UP BARS.

Bulletin No. 29, University of Illinois.

Span 6 ft.; per cent. of metal, 1.25; age, 2 months; third point loading.

No.	Total Load at First Diagonal Crack, lbs.	Vertical Shearing Stress at First Diagonal Crack, lbs. per sq. in.	Stress in Longitudinal Reinforcement at First Diagonal Crack, lbs. per sq. in.	Maximum Applied Load, lbs.	Maximum Vertical Shearing Stress, lbs. per sq. in.	Maximum Stress in Longitudinal Reinforcement, lbs. per sq. in.
532.1	18,000	139	26,200	20,600	159	30,000
532.2	18,000	140	26,300	18,000	140	26,300
554.1	.....	.....	.....	17,900	139	26,200
554.2	.....	.....	.....	17,500	136	25,600
553.1	24,000	183	34,600	28,800	220	41,700
553.2	24,000	186	34,800	31,000	240	45,000
Averages	21,000	162	30,500	22,300	172	32,400

pression portion of the beam, in consequence of which this region remains intact and behaves somewhat as the corresponding portion of a stressed homogeneous beam.

In *Bulletin No. 29,* University of Illinois, is contained a vast amount of data relating to a large number of tests on reinforced concrete beams tested with a view to throwing some light on the question of resistance to web stresses. With a view to obtaining therefrom some information relative to the comparative strengths of beams in which the only variable would be the methods of web reinforcement, the writer selected three series of beams where dimensions, span, age, mixture and method of loading were all the

same but where the first had no web reinforcement, the second had the main rods bent up in places and the third had stirrups of variable spacing. With the aid of a slide rule Tables I, II and III have been constructed. In Tables I and II all failures were by diagonal tension; in Table III, all but three, viz., 232.1, 232.2 and 233.1 failed in the same way. The three named failed in tension. It will be seen that the average strengths are as 1:1.19:1.74. Since the average yield point of the reinforcement was 58,000 lbs. per sq. in., it would seem as though the full strength of the reinforcement had not been developed in any of the tests quoted. The résumé is an argument in favor of web reinforcement, in the form of stirrups as well as the bending of the longitudinal rods.

Another instance which shows the efficiency of stirrups in increasing the strength of reinforced beams, and in signally altering the method of failure, is afforded through the tests of Morton Owen Withey, University of Wisconsin, series of 1906 and 1907. These beams were 8 x 11 ins. x 12 ft. span, and Nos. T_1, T_2, V_1 and V_2 of the 1906 series were duplicated by W_1, W_2, X_1 and X_2 of the 1907 series in every particular except that the latter four were provided each with 20 stirrups of ¼-in. round stock in addition to the longitudinal reinforcement, part of which was on the compression side and part on the tension side. The latter in each case was equal to 2.9 per cent. of the section. While the first series are reported as failing in diagonal tension, the others failed through tension in the reinforcement. The results are:

Beam No.	$\frac{M}{bd^2}$	Kind of Failure.
T_1	774	Diagonal tension
T_2	1003	" "
V_1	1884	" "
V_2	1008	" "
Average	917	
W_1	1060	Tension
W_2	1085	"
X_1	1043	"
X_2	1085	"
Average	1068	

The second average, it will be observed, is some 17 per cent. larger than the first. The additional metal in the second series will amount to practically .28 per cent. of the beam. At the ratio of 50 to 1 for the cost of reinforcement to cost of concrete, volume for volume, it will be seen that in these series, an addition of 17 per cent. to the strength has been secured through employing stirrups representing only 4½ per cent. additional outlay. Whether fewer stirrups or lighter ones would have accomplished the same purpose was of course not ascertained.

TABLE III.—BEAMS WITH STIRRUPS.

Bulletin No. 29, University of Illinois.

Span 6 ft.; per cent. of metal, 1.25; age, 2 months; third point loading.

No.	Total Load at First Diagonal Crack, lbs.	Vertical Shearing Stress at First Diagonal Crack, lbs. per sq. in.	Stress in Longitudinal Reinforcement at First Diagonal Crack, lbs. per sq. in.	Maximum Applied Load, lbs.	Maximum Vertical Shearing Stress, lbs. per sq. in.	Maximum Stress in Longitudinal Reinforcement, lbs. per sq. in.
231.1	25,260	194	36,600	33,000	251	47,800
231.5	18,000	137	26,000	35,800	272	51,800
231.6	20,000	152	28,900	31,800	242	46,000
232.1	28,000	207	40,500	38,000	281	55,000
232.2	24,000	182	34,800	40,300	306	58,200
232.5	17,000	131	24,800	22,000	170	32,000
232.6	19,000	146	27,600	25,000	192	36,300
233.1	27,000	205	39,000	43,300	328	62,500
233.2	20,000	152	28,800	34,700	264	50,200
233.5	14,000	107	20,300	34,000	259	49,300
233.6	14,000	107	20,400	29,300	224	42,500
233.7	23,000	176	33,400	27,200	208	39,500
Averages	20,800	158	30,100	32,800	250	47,600

It is observed also that while the maximum tensile stress in the longitudinal reinforcement computed from the known bending moment, was in the first four beams well below the elastic limit of the material, the stress for the second four was correspondingly above it.

The purpose of web reinforcement is not, as the writer sees it, necessarily to prevent failures in diagonal tension. That is incidental to its main function which is to prevent the failure of beams long before the reinforcement in tension or the concrete in

compression have been stressed to a degree anticipated by the designer. If failures in diagonal tension occur when the longitudinal reinforcement has been stressed to, or nearly to, its elastic limit, and when the concrete is about to fail in compression, it does not seem that additional web reinforcement is likely to be very efficacious. Indeed, it has seemed to the writer that some of the failures reported as diagonal tension failures are of such a character that it is exceedingly difficult to state precisely what the type of failure really is.

Now the theoretic treatment of this problem, as has been seen, is an exceedingly difficult one chiefly because a stressed reinforced concrete beam, especially after incipient rupture of the concrete has begun, is scarcely amenable to the principles of stress which are applicable to beams of homogeneous construction. At all events, the writer is willing to grant that web reinforcement, in the light of tests, has vindicated its purpose; but, believes that its proportionment and disposition in the beam will probably be arrived at through empirical rather than through theoretical investigations. There is probably enough experimental data available from European and American sources by which to arrive at a working basis, if someone possessing genius for analysis and the necessary time, were to undertake its discovery. It does not appear that as yet, it has been done.

In the construction of T-beams, the stirrup, vertical or inclined, performs a very useful function in bonding together the web and the flange. Because of the exigencies of construction and the possibility that, in spite of explicit requirements regarding it, the slab may not have been cast monolithic with the web, the stirrups will serve to bind the two parts of the structure together. There is without doubt, sometimes, a plane of weakness between the slab and web which the stirrups, acting in shear solely, are very efficacious in rectifying.

It would appear as though the stirrups should be more closely spaced toward the ends of a uniformly loaded beam than toward its center. Theoretical considerations would suggest that for such a case, since the shearing stresses are intensified toward the extremities of a beam, the tendency to premature failure by diagonal rupture will be greater there than toward its center. This tendency

is negatively illustrated by the fact that on beams tested by two point loading, and reinforced against failure uniformly from support to load point, the so-called diagonal tension failures occur apparently anywhere and everywhere in the portion subjected to constant shear.

There seems to be justified the recommendation of the Joint Committee on Concrete and Reinforced Concrete, that vertical stirrups be figured according to the formula—

$$P = \frac{Vs}{d^1}$$

where P = total stress taken by one stirrup,
V = that part of the total shear taken by the metal,
s = the spacing of stirrups, and
d^1 = the distance from center of tension to center of compression.

A modified form is also proposed for inclined stirrups. A large number of tests on beams of good concrete without web reinforcement of any kind, shows that failures in diagonal tension occur when the maximum shearing stress reaches about 140 lbs. per sq. in. The recommended value of 40 lbs. per sq. in. as a maximum working value, for such beams seems very moderate indeed. With ample web reinforcement, this may with safety be raised to 150 lbs. per sq. in.

The bending up of horizontal bars, while helpful is generally inefficient since they are usually too large in section and too few in number to perform their function economically. Web members should preferably be numerous and well distributed throughout the mass they are intended to restrain. Wire fabrics and expanded metals have given good satisfaction when used as web reinforcement. The use of old nails, as has been somewhere proposed for reinforcing concrete, has one point in its favor in that the reinforcing elements are numerous and of small section and pervade the entire mass whose integrity they are intended to safeguard. A desirable system would be one where, as in a truss, the reducing section of the members of the web system, as one proceeds toward the center, is accompanied by an increasing section in the chord members.

It is most necessary to require that every provision be made to secure a satisfactory connection of stirrup to longitudinal reinforcement below, and to the concrete in the compressive region above. The longitudinal reinforcement should rest in the U of the stirrup, if it be not a part of the main rod, and provision against slipping should also be made. The writer believes that nothing is quite so satisfactory as a rigid connection between the web and longitudinal members. Moreover, he believes that the efficiency of stirrups would be very much increased by their having in the majority of cases a rigid connection with a top chord as well as with the bottom bars. He has been led from his own observations on the behavior of reinforced concrete beams under test, to suspect that many of the reported diagonal tension failures are due to the slipping of the web members out of the compression portion of the beam, and has been puzzled to understand sometimes the reported tension failure of web members not specially secured.

PROBLEMS ENCOUNTERED IN CONSTRUCTING THE AZISCOHOS STORAGE DAM.

By Seth A. Moulton.*

The new reservoir, which will be created by the construction of the Aziscohos Dam, will be a unit in the comprehensive system of water storage which has been developed at the head waters of the Androscoggin River. The scope of this paper will not permit a detailed description of the entire storage project; but it is necessary, for a thorough understanding of the subject in hand, that a brief history be given of the original enterprise and its relation to the work now in progress, which this paper purports to describe. For convenience of presentation it will be subdivided into six general headings which will be discussed in the order named, as follows:

1. History
2. Organization
3. Transportation
4. Design
5. Logging
6. Construction

1. History.

Referring to the accompanying map (Fig. 1), it will be seen that the present reservoir system comprises the natural lakes and ponds in that section of northwestern Maine commonly known as the Rangeley Lake region; including Rangeley Lake proper, Kennebago, Cupsuptuc, Mooselucmaguntic, Mollychunkamunk, Welokennebacook and Umbagog Lakes, together with several small ponds at the head waters of tributary streams. The discharge from these lakes is controlled by substantial, rock-filled log crib dams constructed about 32 years ago at the outlets of Rangeley, Mooselucmaguntic, Welokennebacook and Umbagog Lakes. The waters impounded by these dams are utilized to increase the low water

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flow of the Androscoggin River for the benefit of the various mills that obtain water power from the Union Water Power Company's

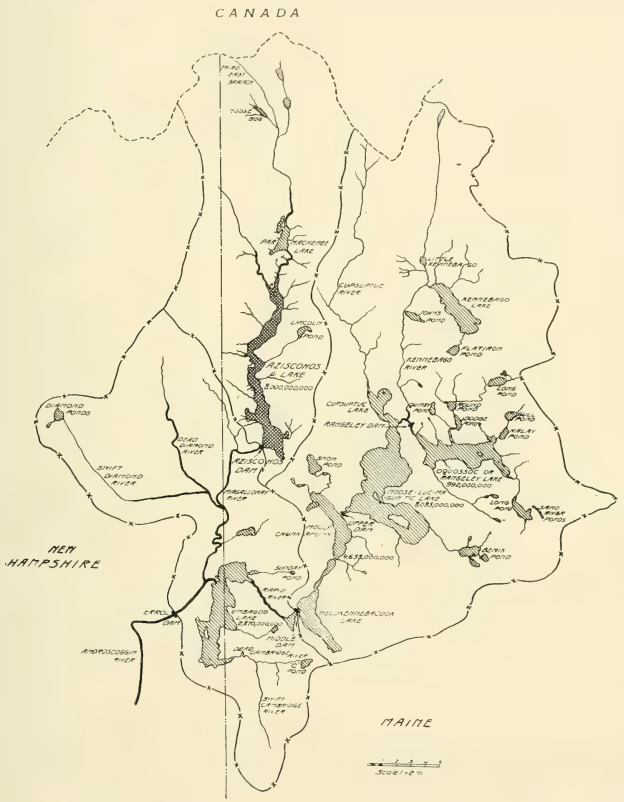

FIG. 1.—MAP OF RANGELEY LAKES AND DRAINAGE AREA.

canals in Lewiston, Maine. The rapid growth of manufacturing on the Androscoggin River, simultaneous with the development of the falls, between the outlet of the lakes and the City of Lewis-

ton, combined with the additional demand for water power in Lewiston and at other points on the river, has made the present storage supply inadequate for furnishing the required low water stream flow.

For the past seven years, Mr. Walter H. Sawyer, the Agent and Engineer of the Union Water Power Company, has been constantly studying the storage problem, ultimately reaching the conclusion that it was entirely feasible, and of immense value to all water power users on the Androscoggin River, to create a new lake at the head waters of the Magalloway River, a tributary stream discharging into Umbagog Lake. To form this new lake it was necessary to construct a new dam at the head of Aziscohos Falls, near the site of an existing crib dam which had been constructed by lumbermen and was used for logging purposes only.

The present reservoir system has storage capacity for about 17 billion cu. ft. of water and the new basin a capacity of about 8 billion cu. ft., or an additional supply equal to practically 50 per cent. of that now available. The value of the increased storage is best defined by stating that the Aziscohos Lake will furnish a volume of water sufficient to operate all the mills in the City of Lewiston for a period of 72 days, without supply from any other source.

2. Organization.

Before proceeding with a description of the work proper, brief mention should be made of the construction organization, as the methods adopted for its perfection produced an exceptionally efficient and frictionless machine which proved to be well adapted to the requirements, anticipating the needs of the component parts of the work in a manner far exceeding the most sanguine expectations.

Owing to the prolonged delay in the selection of the type of dam, it was considered impossible to prepare sufficient detailed plans and specifications, from which satisfactory contractors' bids might be obtained, in season to proceed with the work during the year 1910. On account of this condition, the Androscoggin Reservoir Company instructed the engineering firm of Sawyer and Moulton to proceed with the work on the "force account" basis,

with orders to commence at once to accumulate the necessary apparatus and supplies for its execution. This decision was reached on February 15, 1910, allowing, under the most favorable weather conditions, a period of only 8 weeks in which to collect and deliver to the dam site an exceptionally large contractors' equipment; comprising a 1,200-ft. span, 5-ton cable way, hoisting engines, derrick fittings, sluicing pumps, concrete mixers, stone crushers and such other heterogeneous materials as are ordinarily required for work of this description.

The task seemed almost impossible, involving, as it did, the purchasing and transporting of 250 tons of material within so short a time. Fortunately, success attended the attempt to obtain and deliver the necessary outfit; although at one time the situation seemed dubious, owing to the fact that an early thaw set in before the cable was delivered at the railroad terminal, and the snow had practically disappeared from a portion of the highway (over which a streak of slush and mud indicated where the road should have been). The cable was started on wheels but met with disaster when transported but a few miles; and from thereon brute force alone accomplished the dragging of the cable on an improvised timber sled, through the mud, to the dam. The cable was a piece of apparatus absolutely essential to the efficient conduct of the work, accordingly no expense was spared in the endeavor to deliver the same, and after 3 weeks of gruelling effort, with the aid of 20 horses it was successfully landed at its destination. The engineers were confronted with the problem of economically constructing a work of first magnitude in a wild and undeveloped country, with the contingent difficulty of maintaining a large community remote from its base of supplies.

To accomplish the above, careful detailed analysis was made of each separate item of the work, the results of which are assembled and graphically depicted in the Organization Diagram (Fig. 2). This diagram is of particular interest on account of the variety and complexity of the work which it covers and also from the fact that its deductions made possible proper provision for temporary buildings and construction equipment; while its existence, in connection with written field instructions, which were issued to the several officials, practically eliminated any question regarding the respective authorities of the various employees. Throughout

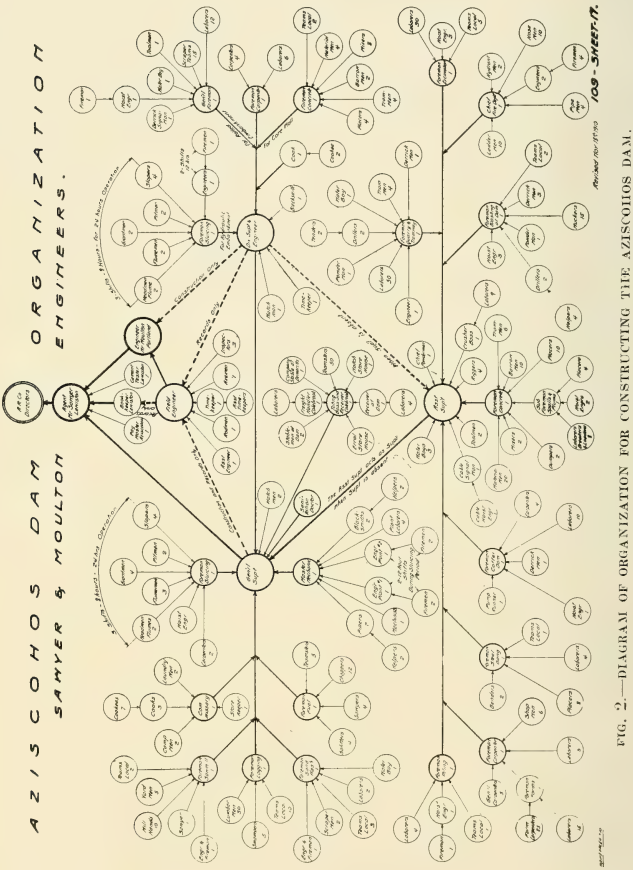

FIG. 2.—DIAGRAM OF ORGANIZATION FOR CONSTRUCTING THE AZISCOHOS DAM.

the construction work has proceeded smoothly, without delay, other than that occasioned by natural conditions; in consequence, since the commencement of the concreting on July 13, 1910, an average of 100 cu. yds. per day has been placed; a very satisfactory accomplishment, considering the character of the design.

Simple and efficient methods were adopted to keep the engineers in constant touch with the work, consisting of a progress diagram (Fig. 3), copies of which were filled out daily by the field engineer and forwarded to the offices of the agent and engineers, in connection with a complete system of daily reports tabulating in detail the day's work and the distribution of the labor. From these daily reports, the cost keeper, in the engineers' office, compiled the unit cost of the several parts of construction, transcribing same to cost charts which showed, by plotted curves, the daily unit cost and the average unit cost of each specific class of work up to the date in question. This has enabled the engineers to keep constantly informed as to the condition and cost of the work without the laborious perusal contingent on examining numerical cost sheets.

3. Transportation.

The remote location of the dam made it apparent that the most difficult problem to be solved would be that of transportation. The work is situated about 38 miles from the nearest available railroad station at Colebrook, N. H. The intervening highway being of the ordinary country type is, owing to adverse climatic conditions, practically impassable during from 6 to 8 weeks in the ordinary spring season, or from about April 1 to the middle or last of May, and it is useless to attempt to haul a load of any magnitude during this period. Even during the most advantageous summer weather sections of the roads are in poor condition for heavy teaming due to the nature of the road material and the effect on same of the frequent showers combined with almost continuous cloudy weather.

During the first two months of the ordinary winter season "toting" can be carried on more advantageously than at any other time; but after the first 6 to 8 weeks of sledding, the fall of snow becomes so great that it is very difficult to keep the teams on the

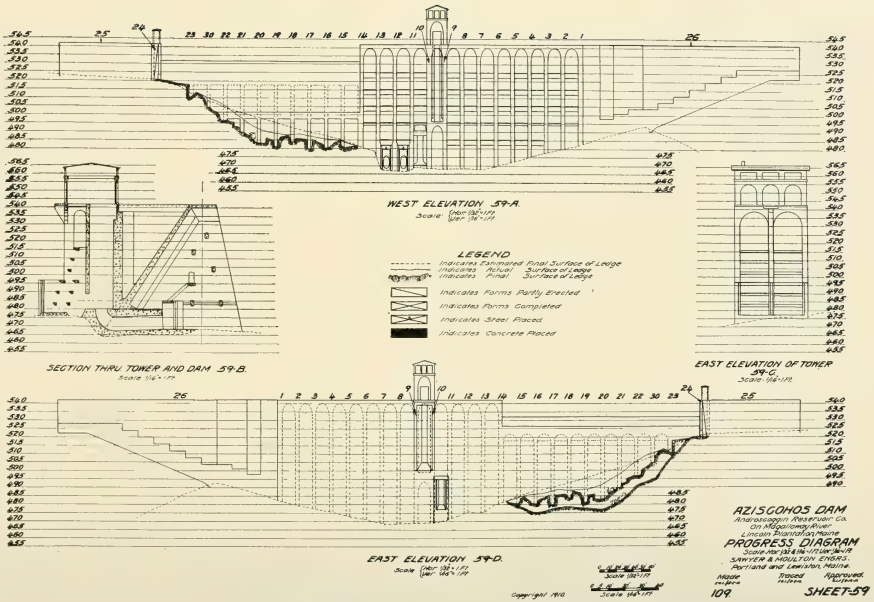

FIG. 3.—PROGRESS DIAGRAM FOR DAILY RECORD.

built-up trodden section of the roadway unless special precaution is taken to provide proper turn-outs and to keep the surface of the hard road well below the level of the surrounding snow.

It was obvious that whatever type of dam was adopted it would be necessary to haul material at least during one winter and one summer season, and possibly during two winter seasons. The toting situation necessitated that the volume of material to be hauled should be carefully predetermined in order that it might be delivered in anticipation of the needs of the work.

Serious consideration was given the advisability of employing traction engines with trailer sleds for winter toting and motor trucks for summer toting; but after an exhaustive study of the conditions it was decided to haul the materials with "tote" teams, the method of transportation common to this section of the country. The country passed through being very sparsely settled, it was with difficulty that provision was made for sufficient accommodations at the intermediate stopping points, and carefully prepared timetables were established to which, barring accidents, the teamsters were compelled to adhere; neglect to obey the toting superintendent's orders regarding stopping points being considered sufficient reason for the discharge of the offending teamster, unless ample proof could be furnished for the necessity of disobedience.

All material was hauled by the hundred-weight and each operator was held responsible for the material in his custody from the time same was delivered to him at the railroad station until deposited in the company's store houses at the dam site. A comprehensive system of triplicate waybilling and checking was devised whereby exact but simple records were kept of the material hauled, at both the Colebrook and the dam store houses, one waybill being retained by the teamster as a voucher.

The round trip of 76 miles from Colebrook to the dam and return is made in 4 days, each team making two complete round trips without rest and laying off the ninth day. 4-, 5- and 6-horse teams were employed, averaging 4.5 horses to a team, and by a proper arrangement of the time schedule a maximum total of 180 horses could be accommodated on the road, making a total of 40 teams which, on the basis of 4 days to a trip, gave an average of 10 teams arriving at the dam during each day of the toting season.

During the best period of toting, or for the 6 weeks from December 1 to January 15, the average loads were

4-horse	team	6,650	lbs.
5	"	8,600	"
6	"	10,400	"

The average load per horse being 1,680 lbs.

During the best period of summer toting:

4-horse	team	5,750	lbs.
5	"	None working	
6	"	8,400	lbs.

The average load per horse being 1,430 lbs.

During the most adverse condition of toting:

4-horse	team	4,800	lbs.
5	"	6,200	"
6	"	7,200	"

The average load per horse being 1,220 lbs., or 20 per cent. less than could be hauled during the best sledding.

The total average load hauled per horse was 1,460 lbs., for which the toting contractors received 65 cts. per hundred-weight, as a flat rate for all seasons, making $2.11 the average earning per horse per day. The average cost to the company for materials delivered to the dam is 85 cts. per hundred-weight, the additional 20 cts. being due to overhead charges, including the cost of superintendence, bookkeeping and store house expenses.

The preliminary investigation of the toting problem indicated that the maximum volume of material which could be transported during a winter sledding season would not exceed 5,000 bbls. of cement or a gross weight of two million pounds, if the usual methods of procedure in vogue prior to this time were employed. This amount was so obviously insufficient, even for the preliminary stages of the work, that special effort was made to overcome each interfering detail in order that a maximum tonnage might be moved during the first season. Consultations were held with teamsters and toting contractors, who had devoted their entire lives to hauling supplies into the woods for the lumbermen, and no encouragement was obtained from these sources as to the probability or possibility of exceeding the previously stipulated quantity of 5,000 bbls. of cement, or its equivalent of two million pounds.

In spite of the discouraging prospect, the situation was carefully analyzed and a system of operation adopted which has had no material change since its inception.

The prime factor limiting the quantity of the material which can be transported is the topographical condition of the country over which the road is constructed. The highway passes over a divide in the Dixville mountains at a distance of about 12 miles from Colebrook and this portion of the road has a constant up-hill grade, necessitating much lighter loading of the teams than would be otherwise essential, as the road between Dixville Notch and the dam is comparatively level. To obviate this condition an intermediate store house was erected beyond the height of land in the Dixville Notch to which relay teams hauled materials from Colebrook. This arrangement permits the through teams to start with small loads, increasing same at the Dixville store house to the full capacity of the equipment, if the roads beyond are favorable.

A second intermediate store house was provided at Errol, N. H., about 20 miles from Colebrook to which relay teams delivered materials. This provision permits the hauling of maximum loads from Errol to the dam even in the event of bad roads between the Dixville store house and the above mentioned village, a likely condition owing to the exposed situation of this region.

That the system was successful is shown by the results of the first season, from December 1, 1909, to February 10, 1910, a period of 72 days, 11,000 bbls. of cement, together with other miscellaneous supplies amounting, in total, to 4,500,000 lbs., were delivered to the dam site, more than 100 per cent. in excess of the prophesied quantities. From the commencement of the toting on December 1, 1909, until November 1, 1910, a period of 335 days, the total amount conveyed, as shown by the teamsters' records, was 7,250,000 lbs.

The consideration of the above mentioned facts reveals the obvious need of a design requiring a minimum weight of toted materials.

4. Design.

The reconnoissance surveys and preliminary borings made during the summer of 1908 indicated that the best location for the dam would be at a point in the river valley about 250 ft. below

the old crib structure, as shown in Fig. 4, which is a plan indicating the relative positions of the old and new dam, the contiguous laborers' settlement, and the general arrangement of the complete construction plant.

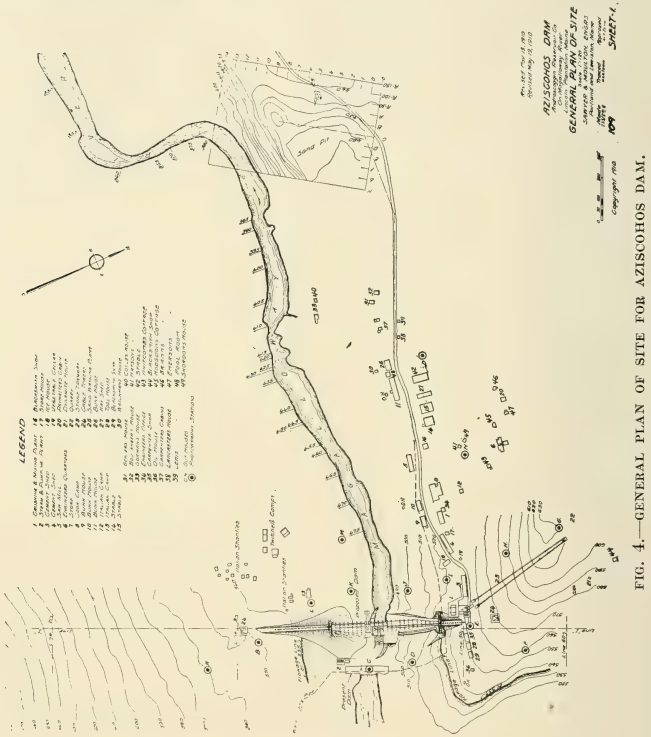

FIG. 4.—GENERAL PLAN OF SITE FOR AZISCOHOS DAM.

The conditions controlling the selection of a proper design for the dam, places this work in a class peculiar to itself, for in addition to the limitations imposed by the transportation problem, the logging situation presents structural difficulties rarely, if ever, before encountered in constructing a storage dam. No claim is

made that the elemental scheme of the adopted type is original, as the idea has been closely paralleled by several projected prototypes; although to our knowledge none of these have been carried to completion. The structure contemplated for the Pioneer Power Plant* has a very similar cross-section.

Fig. 5† shows a sketch of the dam as it will appear when completed. The selection of the cellular construction was made after careful consideration of all the conditions involved; its final acceptance being based on the fact that it contained a minimum quantity of material with maximum stability, an essential property for this particular dam; but the writer wishes to record at this time the general statement that in his opinion with less adverse conditions of transportation, and of auxiliary complications, no economical reason can be offered for its adoption. Certainly the engineering duties are much more arduous and taxing than they would be for a design of more simple composition.

The general dimensions of the dam are as follows:

Total length from end to end of core walls	881 ft.
Maximum neight from crest of main dam to ledge foundation	78 "
Length of main dam	265 "
" spillway	236 "
Height from surface of water in river to level of water surface at full lake	60 "
Crest of main dam above full lake level	7 "
Top of embankment above full lake level	12 "
Maximum high water above full lake level	6 "
Depth of draft from reservoir	47 "

A typical cross-section of the main dam taken centrally between buttresses is shown in Fig. 6. This portion of the structure consists of fourteen 4-ft. thick buttresses spaced 20 ft. on centers. The tops of the buttresses being approximately 13 ft. 6 ins. wide with slopes on the upstream and downstream faces respectively of 7 9/16 ins. and 2½ ins. per ft. of height, these proportions being found ample to insure the stability of the structure, with conservative unit pressures in the buttresses. Arched slabs spanning between the piers form the deck and crest, the intrados of the arches having a radius of 8 ft. The section *B-B* shows the deck slabs near the top of the dam. It will be noticed

Transactions, Am. Soc. C. E., Vol. XXXVIII.

†Acknowledgment is made to the *Engineering News* for the use of Figs. 5-7 and 9-13.—ED.

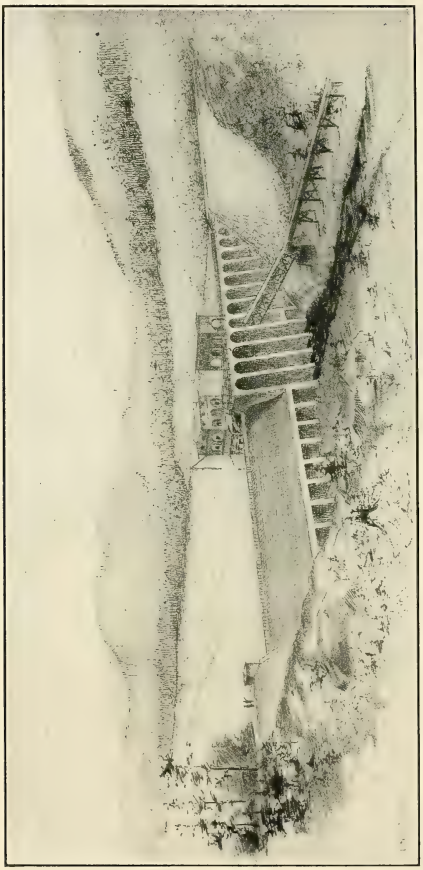

FIG. 5.—SKETCH SHOWING THE AZISCOHOS STORAGE DAM.

that the first 3 ft. of the intrados of the arch ring are cast on a bevel instead of circular, and integral with the buttresses. This was done to simplify the form construction, details of which will be later explained. It will also be seen by reference to the section that the haunches of the arches are cast with solid spandrels, making the water face of the dam a smooth slope with a batter from the vertical of 8 ins. per ft. The crown thickness of the arch slabs at the crest of the dam is 2 ft., and at the lowest point approximately 4 ft. Referring to sections *A-A* and *B-B*, particular attention is called to the shrinkage joints provided over the center of each buttress, and the locks in these joints, between the abutting spandrels, which are incorporated to prevent seepage. These locks are staggered at the various vertical stopping points in the manner indicated on section *D-D*.

Rectangular reinforced concrete struts, or beams, are provided between the buttresses to insure lateral stability (Fig. 6).

A typical cross-section of the spillway is shown in Fig. 7. The spacing of buttresses, the upstream profile of buttresses and the deck slab for this work are identical with the main dam construction. A mass of cyclopean masonry is provided for the spillway crest to resist the action of the ice pressure previously mentioned, and an overflow apron is constructed on the downstream face to deflect the water from the buttress bases. Owing to the existence of the crest and apron the stiffener beams employed between the main dam buttresses are omitted between the spillway piers.

All of the arched slabs are reinforced with deformed steel bars, sufficient metal being used to prevent cracks at the crown of the arches and to concentrate the effect of temperature and shrinkage stresses at the spandrel joints.

The deck slabs for the spillway were completed in August and have consequently been finished about 4 months, a period sufficient to develop the greater part of the shrinkage deformation, during which time there has occurred a temperature change of 90 degrees F. Apparently the shrinkage reinforcement has successfully accomplished the desired results for all of the completed joints have opened equally about 1/16 of an inch throughout their entire height on the upstream face of the dam, these cracks diminishing from the face until they become invisible at the arch springing lines on the buttresses.

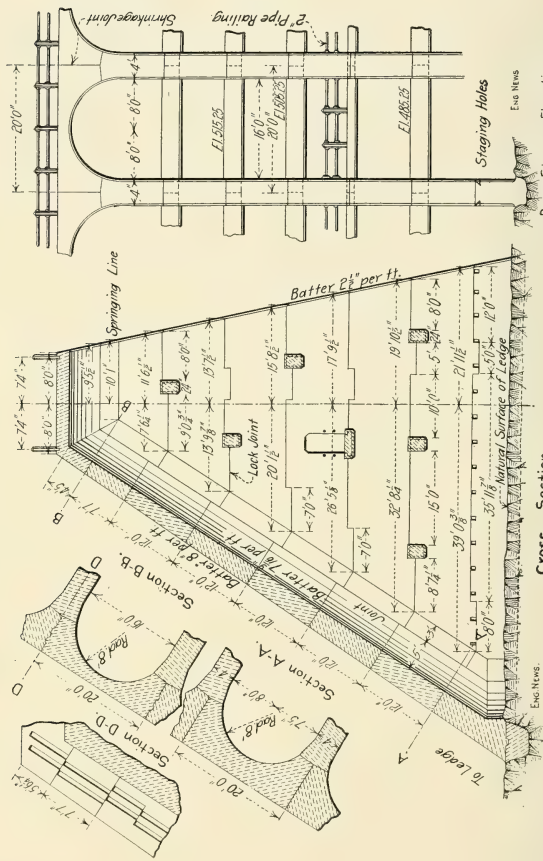

FIG. 6.—TYPICAL CROSS-SECTION OF MAIN DAM, CENTRALLY BETWEEN BUTTRESSES.

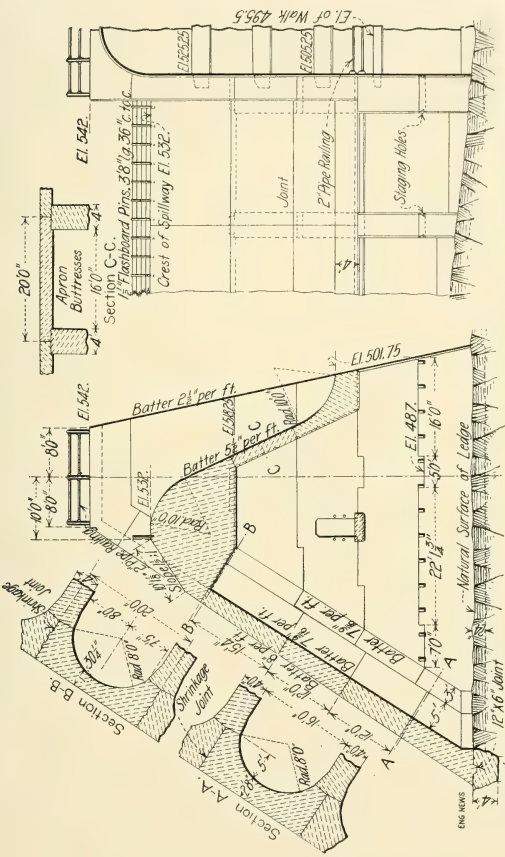

FIG. 7.—TYPICAL CROSS-SECTION OF SPILLWAY, AZISCOHOS DAM.

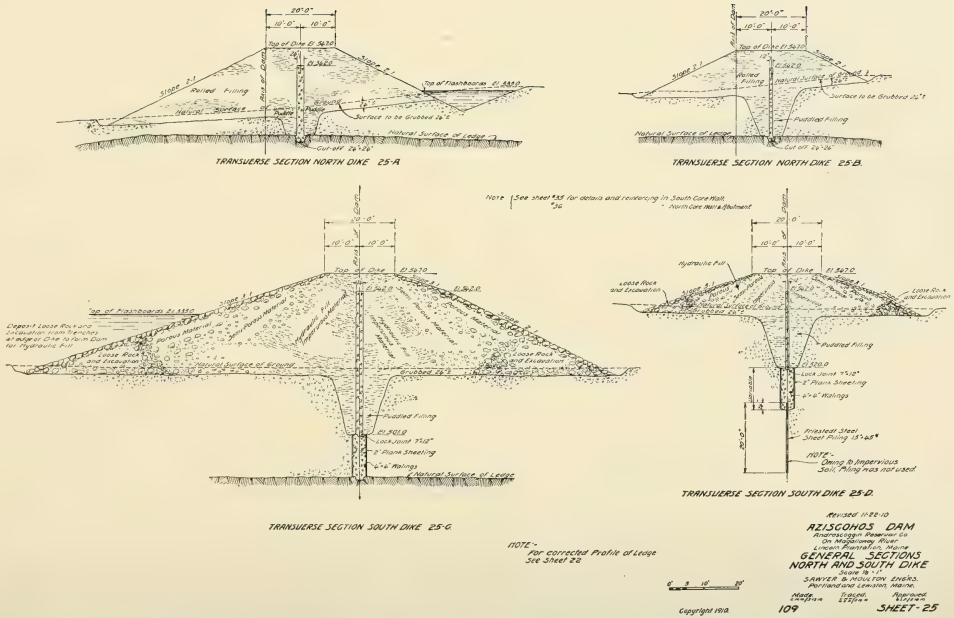

FIG. 8.—GENERAL SECTIONS OF NORTH AND SOUTH DIKE, AZISCOHOS DAM.

The heavy spillway crest is reinforced with channel beams, provided more especially to support the forms and heavy weight of the concrete during construction. The overflow apron is reinforced with sufficient metal to withstand the impact due to a maximum freshet overflow.

The different grades or mixtures of concrete used in the several portions of the structure have proportionate parts as follows:

Class.	Cement.	Sand.	Stone.
A	1	2	4
B	1	2½	5
C	1	3	6
D	1	4	7
E	1	2½	5 with spalls.
F	1	3	6 "
G	1	2½	5 cyclopean.
H	1	3	6 "

The varying aggregate dimensions for the above were defined by exponent figures in connection with the class letters; for example: "A-¾" indicates a 1-2-4 composition having a graduated crushed stone aggregate, with sizes varying from the fine material rejected by a ¼-in. dust screen up to and including the fragments that pass the perforations of a ¾-in. diameter screen. The use of stone dust was permitted by considering same as sand, and reducing the amount of sand proportionately.

The class A concrete is employed in the construction of the gate and logway buildings, and for those portions of the structure that require slender cross-sections of maximum strength. The arch deck slabs and all concrete placed below water level are composed of class B, E or G concrete. The two latter grades are used in situations where the cross-sections will admit the interspersion of hand or derrick placed, thoroughly clean, stone.

Above water level the buttresses are constructed of class C, F or H concrete, the permissible grade depending on the area and shape of the cross-sections. Class D concrete is only used for heavy mass work of minor importance, where bulk alone is needed.

The southerly end of the main dam is embedded in an earth embankment with a reinforced concrete diaphragm, or core wall, extending from the last buttress into the solid material of the natural bank a sufficient distance to insure a practically impervious

cut-off. This construction was adopted on account of the extreme depth of the ledge at this point, as is shown on the cross-section (views 25-C and D, Fig. 8).

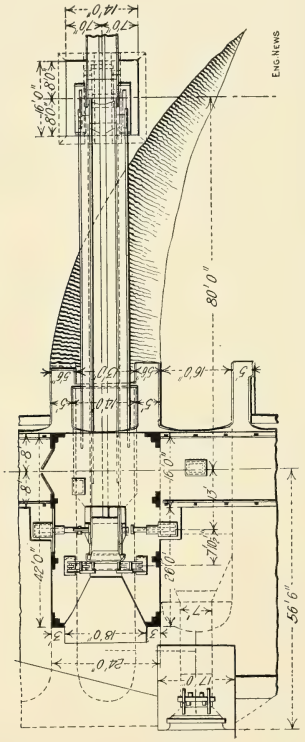

FIG. 9.—SECTIONAL PLAN OF LOGWAY AND WASTEWAY, AZISCOHOS DAM.

It was originally intended to place the embankments by the hydraulic sluicing process, and apparatus was obtained for the purpose; but the exceedingly hard formation of the borrow pit material made it uneconomical to employ this method, on account

of the expense contingent on pumping a large quantity of water under high pressure with very low efficiency of earth displacement.

Test pits indicated that after the hard surface soil had been penetrated, water bearing subsoil would be encountered; accordingly provision was made for driving steel sheet piling under the south core wall for a considerable distance from the main dam buttress, but this was found to be unnecessary.

5. Logging.

Mention has been previously made of the logging contingency, but for a full appreciation of its effect on the work a brief description of the problem is essential, in order that the adopted methods for solving this difficulty may be understood.

The entire water shed appurtenant to the Aziscohos Lake is heavily wooded and will undoubtedly be for all time a timber growing country, owing to its location and mountainous character. Investigation of the lumbering situation revealed the fact that, without question, an average of 20 million ft. of logs (board measure) will be sluiced annually from the lake during the next twenty years. The river below the proposed dam has a fall of 200 ft. in a distance of 2 miles, a condition necessitating the flow of a large volume of water to float the logs over the falls and rapids; and for supplying "sluicing head" the existing dam was built.

Records of the water discharged from the old dam during the sluicing season, show that under the most favorable existing conditions 137 million cu. ft. of water are required for each million of logs passed through the dam, with an average annual total of 2,740 million cu. ft., or more than 25 per cent. of the entire storage capacity of the lake. By improving the river channel with wing dams that will restrict the width of the water ways at abrupt pitches, it will be possible to reduce the average total quantity of water required for sluicing to 1 billion cu. ft. per annum. Ultimately still greater economy of water for log driving will be obtained by constructing a V-shaped flume, or log chute, from the dam to the foot of the rapids, when the average total water required will be reduced to 250,000 cu. ft. per annum.

The total annual average run-off from the water shed is about 10,596 million cu. ft., or roughly 30 per cent. more than the 8 billion cu. ft. essential for filling the lake; but study of the run-off

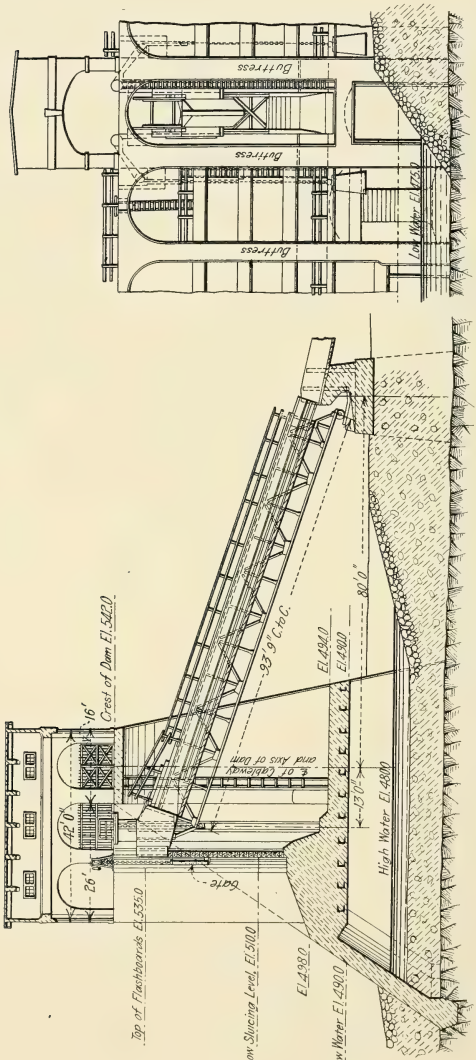

FIG. 10.—SECTION THROUGH LOGWAY, DOWNSTREAM ELEVATION, AZISCOHOS DAM.

records indicates that the lake will not be filled every year. The lake will be at its lowest level during the latter part of March, and will, in ordinary seasons, obtain most of its supply from melting snow in the spring. The log driving season commences as soon as the ice leaves the lake, an event several weeks in advance of the thaws that melt the snow and ice in the woods.

Consideration of the above facts makes it apparent that provision must be made for sluicing the logs through the dam at varying stages of lake level. Thorough study was made of the run-off in relation to the sluicing requirements, resulting in the conclusion that it would be necessary to design a log-way which could be utilized at any lake level between elevation 510.00 and the crest of the flash boards at elevation 535.00, or for a range of 25 ft.

General views of the perfected arrangement for the log sluice, and its accessories, together with the adjacent low head waste gate, which will be used to drain the lake and possibly as an emergency log sluice-way, are shown in Figs. 9, 10, 11, 12 and 13.

The essential features of the log-way are: a vertical slot cutting out the arched face slab between two of the buttresses for a depth of 38 ft. below the crest of the main dam, the adjacent buttresses being of sufficient increased thickness to make the clear opening of this slot 7 ft. wide; a movable steel truss, having a clear span of 93 ft. 9 ins. between centers, supporting a V-shaped flume; and an inclined fixed section of V-shaped spout, or flume, continuing from the end of the truss to a downstream point convenient for discharging the logs into the river.

The raising and lowering of the truss is accomplished with the aid of a 15-ton power traveling crane which is furnished with a double speed hoist for quickly handling the stop-logs and the small section of spout that is set in the stop-log grooves, and overlaps the entrance mouth-piece to prevent leakage.

The swinging truss is suspended at the upstream end between a gallows frame, riding on bronze sliding strips, which bear against the planed surfaces of cast iron guides set in the sides of the concrete buttresses. The truss bearings rotate on a trunnion shaft which is attached to the gallows. A rectangular steel mouth-piece is rigidly fastened to the gallows frame, providing an entrance to the log sluice always at right angles to the stop-logs, with a minimum clearance from same.

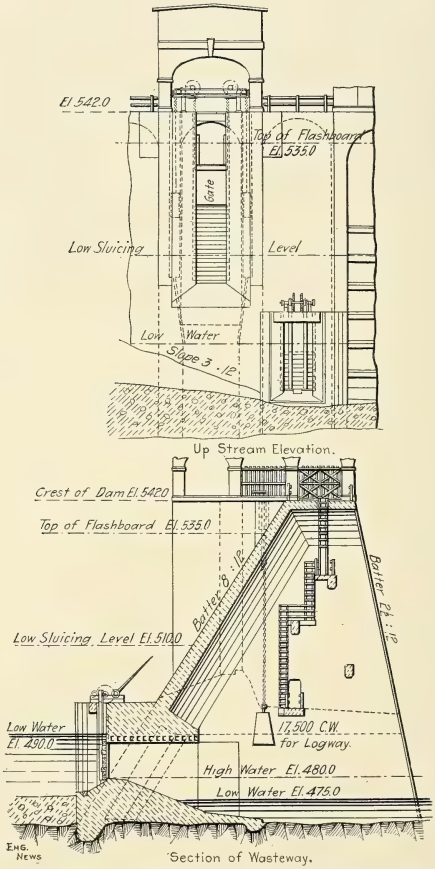

FIG. 11.—SECTION OF WASTEWAY AND UPSTREAM ELEVATION, AZISCOHOS DAM.

On top of the gallows frame headers, a working platform is secured, from which the attending lumberman guides the approaching logs, or manipulates the locking mechanism when changing the position of the sluice. A special system of automatic locks, seating in notches provided in the gallows guide castings, has been devised, whereby the truss always rests on rigid supports, unless the lifting hook is securely fixed in its proper position, thus precluding the possibility of releasing the load from the crane until the truss is safely anchored in the desired position. Chains from the gallows frame are passed over idlers into the adjacent bays of the dam, supporting concrete counter weights which neutralize, to a large extent, the dead load of the truss, relieving the crane proportionately.

The downstream end of the truss is supported on trucks that travel on T-rail tracks having curved profiles of proper shape to maintain a minimum clearance between the overhanging section of steel flume, which protrudes from the truss, and the section of fixed flume that rests on a concrete abutment, designed to resist the shock due to the impact of the rapidly descending logs. The base receiving the end of the truss and the end of the fixed spout consists of a substantial concrete pier founded on bed rock. A heavy cast iron wearing plate is imbedded at the entrance of the fixed spout in the concrete abutment to prevent excessive wear anticipated at this point, owing to an abrupt change in the grade of the flume.

The log-way opening is closed with 12 x 16-in. yellow pine stop-logs bearing in cast iron grooves, situated at the end of the taper approach formed by the converging faces of the concrete abutments. The stop-log castings are imbedded flush with the buttress masonry, and are designed to act as guides for a gravity drop gate which travels in front of the upstream face of the stop-logs. This gate is composed of a structural steel frame encased in concrete, and is suspended on counter weighted chains, controlled by means of a hand operated hoist located on the floor of the log-way house. When the log sluice is operating at the highest levels the drop gate will be submerged a small distance below the top of the stop-logs ready for the instantaneous closure of the log-way in the event of an accident in the flume. At the lower stages of sluicing the gate will be suspended a proper distance above the water level

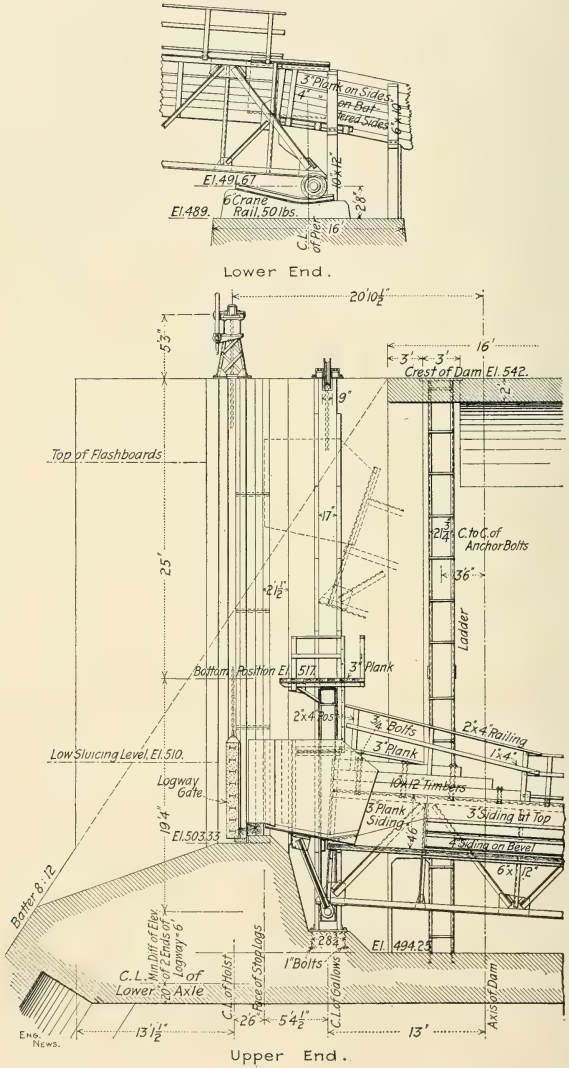

FIG. 12.—DETAILS OF UPPER AND LOWER ENDS OF LOGWAY, AZISCOHOS DAM.

and will be in a position permitting expeditious control. The gate is made of sufficient height to close the log-way with bottom submerged two feet below the top of the stop-logs when they are removed for the full depth of the flume entrance, providing a screen to protect the operator when removing or placing the stop-logs.

A space of 6 ins. is left between the upstream face of the stop-logs and the downstream face of the drop gate. A timber

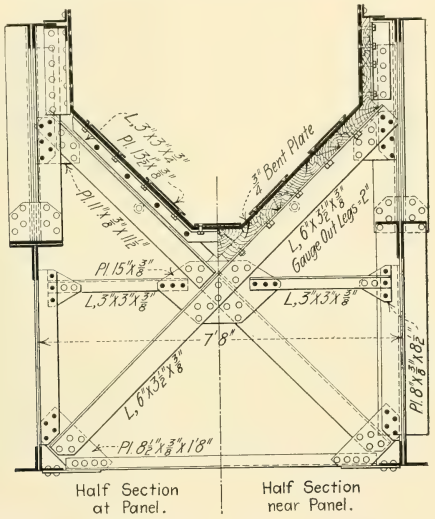

FIG. 13.—CROSS-SECTION OF LOGWAY, AZISCOHOS DAM.

bolted to the bottom of the drop gate closes this space sufficiently to create a pool of quiet water between the stop-logs and the gate, which eliminates the spurting streams that would otherwise occur, seriously handicapping the manipulation of the stop-logs, if this water cushion was not provided.

Before deciding upon the design for the log-way a series of experiments were conducted with large scale working models, and experienced river drivers, called upon to criticise the various schemes, strongly advocated the adopted apparatus, claiming that

its capacity would be limited only by the quantity of logs which could be delivered to the entrance, a feature always the controlling factor regulating the capacity of a log-way. Conservative estimates show that at the higher sluicing levels, under favorable conditions, three million feet of logs, board measure, may be easily sluiced in ten hours, and that the minimum capacity of the outfit will be not less than one million feet of logs in ten hours.

The maximum velocity of the logs at the junction of the movable and fixed flume will be, approximately, 3,000 ft. per minute, and the minimum velocity 1,000 ft. per minute. The swinging spout, which is the elemental feature of this design, is simply an amplification of crude timber structures that have been erected to accomplish the same purpose, under smaller variations of head, by the ingenious Yankee lumbermen of Northern Maine, and is only noteworthy on account of its exceptional proportions.

6. Construction.

The preparatory work of clearing site, building roads, cutting timber for fuel and saw logs and the erection of temporary buildings, preliminary to the actual construction, was commenced on September 1, 1909. The selected form of dam was not determined upon until February, 1910, consequently the tasks accomplished during the winter of 1909 and 1910 were necessarily confined to the execution of such general work as would be required for the erection of any type of structure, and comprised the construction of cement sheds, temporary log sluice, bunk houses and saw mill; also the hauling of cement, lumbering operations and the digging of test pits to determine the location and character of the foundation rock.

By April 15, 1910, the temporary log chute was in full operation, discharging the water released from the gate ways in the old dam, a safe distance below the new work. The cleared plateau on the opposite bank of the river was covered with the stacked and piled fuel cordwood to the extent of 3,000 cords, which is about two-thirds of the total amount of 4,500 cords to be consumed for power. The large quantity of fuel needed is best appreciated by conceiving a continuous pile of cordwood practically 7 miles long. The underbrush was burnt as a precaution to eliminate the risk of forest fires.

Estimates were prepared in the engineers' office showing the probable time limits essential to complete the various sections of the work; compiled from the evidence of test pit data and based upon the diagram sketches of the several stages of operation. The test pits indicated that the over-laying ledge cover was a very refractory material composed almost entirely of compacted, impalpable, rock flour deposited by glaciers. The test pits, in conjunction with a meagre number of borings, indicated that the ledge

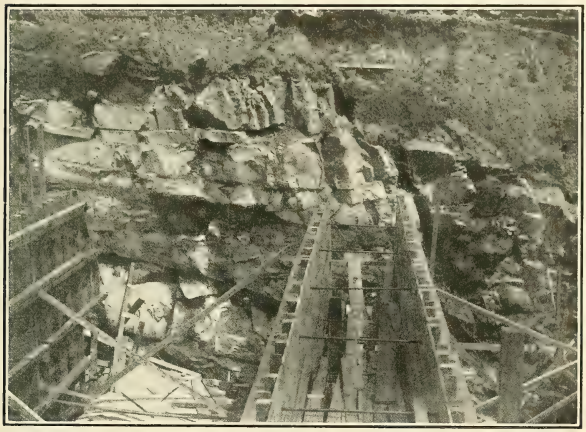

FIG. 14.—VIEW SHOWING SEAMS OF SOFT DISINTEGRATED MATERIAL BETWEEN LAYERS OF HARD GRANITE.

was a very coarse granite formation of medium hardness, providing a foundation entirely suitable for sustaining the contemplated loads. To successfully complete the work in the season of 1910, the progress estimates showed that the concrete plant equipment must be ready for operation, and the spillway excavation be completed, ready for placing concrete, about the first of June. These desired results were attained.

While excavating the ledge trenches, for the buttress footings and the cut-off wall at the base of the upstream face of deck slabs, suspicious and disconcerting seams were encountered, after the

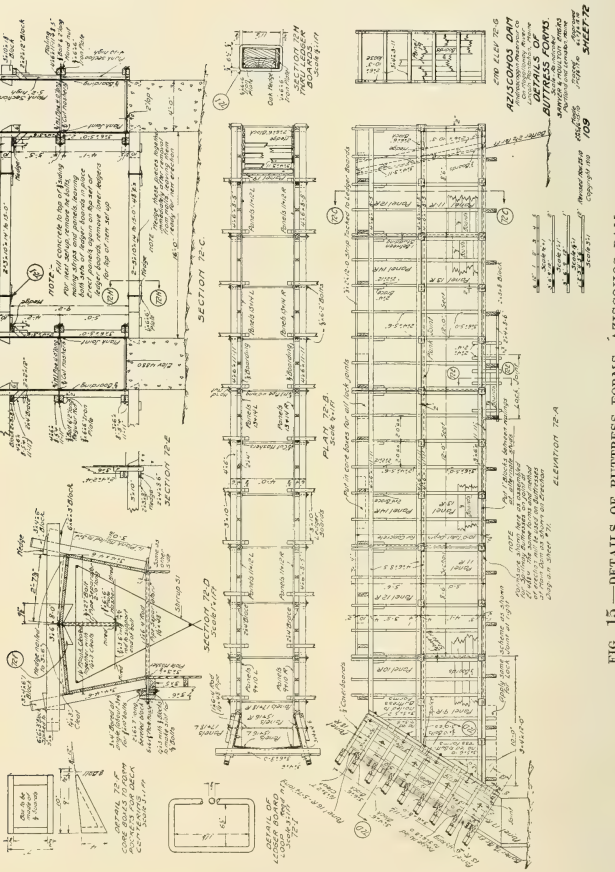

FIG. 15.—DETAILS OF BUTTRESS FORMS, AZISCOHOS DAM

first 2 or 3 ft. of the hard granite surface rock had been removed. These seams appeared to consistently exist at all points excavated and it was realized that the situation was serious. Accordingly immediate steps were taken to determine the extent of the stratification with the ultimate result that it became essential to excavate from 8 to 16 ft. of the surface rock before the laminated formation was sufficiently removed to insure an unyielding foundation. Fig. 14 is a typical illustration of the disintegrated material and clearly shows the soft seams between the hard layers of granite; in the foreground is seen the erected first stage of the buttress forms. The ledge seams were from ½ to 12 ins. deep and were filled with rock flour, intermingled with an occasional layer of disintegrated granite.

The faulty condition of the ledge necessarily upset all time calculations, and precluded the possibility of commencing the concrete work on the first of June as anticipated. This unfortunate predicament seriously handicapped the whole work and delayed the placing of any concrete until July 13. A peculiar circumstance in connection with this incident is the fact that the preliminary diamond drill borings, although admittedly insufficient, gave no indication that there existed any very soft material, except in one instance where a large clay pocket was encountered; but this was on the south side of the stream under the deep natural bank and was attributed to a local fault. The only explanation which can be offered for the above inaccurate representation is that the diamond drill was too small, making it impossible to remove a whole core of any great length, thus deceiving the operators and making them accept the short cores as the result of apparatus limitations.

Forms are always an element of first importance in concrete construction, for they have material effect on both cost and progress, and should, accordingly, be subjected to as careful consideration as any other detail of the design. Realizing the truth of this assertion, and also appreciating the fact that the proposed dam consisted of a series of duplicate parts, it was apparent that a system of interchangeable forms, in standard unit sections, would be not only feasible, but would economically expedite the work, if they were properly constructed. Accordingly complete plans were prepared for the principal form work of the buttresses, arched

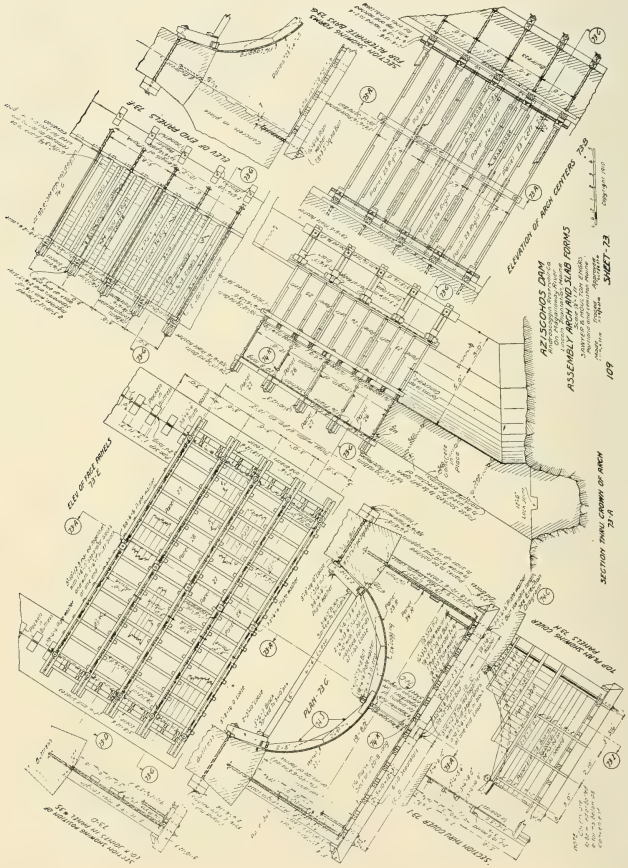

FIG. 16.—ASSEMBLY ARCH AND SLAB FORMS, AZISCOHOS DAM.

slabs and core walls, also for forms having special conditions of loading or that were structurally complex.

The methods of constructing and assembling the buttress forms are clearly illustrated in Fig. 15, and a detailed description of this work is, therefore, unnecessary, but special attention should be called to certain novel features, which practically eliminated all shoring and bracing, at the same time providing rigid adjustable attachments, to which the arch forms were secured. These are: the tie bolts and separators, shown in view 72-D, located at the junction of the straight buttress sections with the beveled nosing, forming arch haunches. The bolt sockets set central with the upstream face of the buttress nosings, and the core boxes, illustrated in view 72-F, which were set in the nosing forms, as shown in view 72-D, for receiving the studdings of the shrinkage joint forms, later described. An especially valuable but not original device, is the extensible double ledger boards, which act as separating struts and spacers for the buttress side panels and also as stringer supports for the working platforms. These ledger boards consist of two 3 x 10-in. timbers, 14 to 15 ft. long, yoked together with two welded iron loops, which are held rigidly in position by means of plates and wedges, as shown in view 72-H. This arrangement provides a stiff beam of variable length that can be easily removed without mutilation, by simply releasing the loop wedges; the ends of the sticks being beveled to facilitate their extraction from the concrete.

The arch forms shown in Fig. 16 appear to be very complex, a difficulty hard to overcome in preparing plans for form work. Careful examination will, however, reveal that this work is much simpler than at first apparent. The essential and radical features of this construction are the methods employed for supporting the intrados arch forms, and for tying the upstream face panels to the buttress concrete and the intrados forms, dispensing with the need for bracing of any description, except the small amount necessary for the temporary support of the forms while they are being placed in position.

The intrados arch panels consist of two circular sections 4 ft. deep, jointed and held in position at the arch crown by specially designed, adjustable tie rod spacers, which pass through the crown of the arches, tying the intrados forms rigidly to the stringer beams

of the upstream face panels. The buttress ends of the arch sections are held in position entirely by the friction grip of abutment blocks, that are securely clamped to the sides of the piers with bolts which pass through the pipe separators cast into the concrete buttresses, and to which attention has already been called. Spreader struts, securely wedged against the abutment blocks, hold the ends of the arch segments tightly against the edge of the buttress bevels. The ends of the segments are supported by 4 x 6-in. sill strips that are seated in shoulders on the abutment blocks, and are rigidly keyed in correct position with wedges.

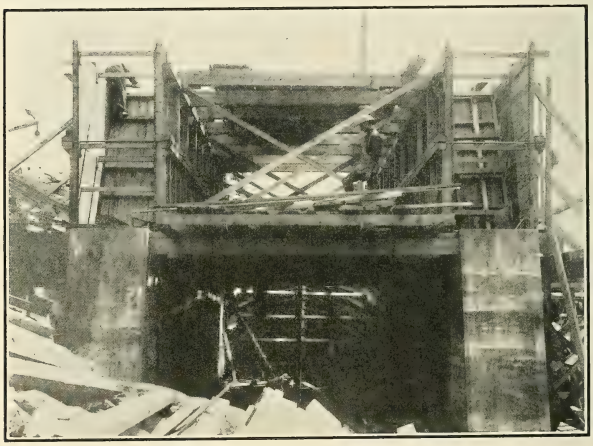

FIG. 17.—BUTTRESS FORMS IN PLACE, AZISCOHOS DAM.

The end panels for the shrinkage joints, central with buttresses, are held by means of the notched recesses provided in the nose of the piers, the top reaction of the studding being received against 3 x 8-in. plates which abut against the shouldered blocking securely spiked to the end of the face slab stringers. The end reaction of the face slab forms is held by anchor rods which are inserted in the pipe sockets incorporated in the buttress concrete. View 73-C clearly shows the above described construction.

To reduce, as much as possible, the opening of the shrinkage

joints between the face slabs, the bays are cast in alternate sections. To obtain maximum benefit from the initial shrinkage of the concrete, a time interval, as long as possible, is allowed to elapse before pouring the concrete in the intermediate bays. Pipe sockets are set in the faces of the first cast face slabs for receiving the anchor bolts that resist the end reaction from the intermediate face form stringers. (See view 73-G, Fig. 16.)

The practical application of the buttress forms is clearly illustrated in Fig. 17. This view is taken below the spillway looking upstream, or east; in the background can be seen an assembled

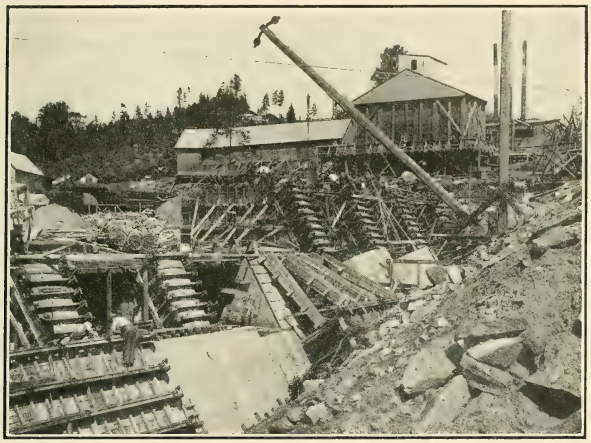

FIG. 18.—SPILLWAY FORMS IN COURSE OF ERECTION AND CONCRETE MIXING PLANT, AZISCOHOS DAM.

section of the intrados arch forms. This picture was taken during the erection of the second lift of the buttress forms. Note the substantial character of the form work, the ample working platforms, especially the clean working space on the upper deck from which the concrete is distributed, the absence of cumbersome, obstructing falsework bracing, and particularly the workmanlike appearance presented by the stripped section of the piers that testifies in favor of this logical method of procedure.

The spillway forms in all stages of erection, the spouts employed for depositing the concrete in the face slabs, the concrete mixing plant with stone bins and adjacent cement shed, and the quarry face at the top of the hill, in the central background, are shown in Fig. 18; which is a view taken on the easterly, or upstream, side of the spillway looking due north. The stone crushers are located in the rear of the stone bins, and the broken stone is delivered from the quarry to the crusher charging platform by means of an inclined, double track, gravity tramway. Again note

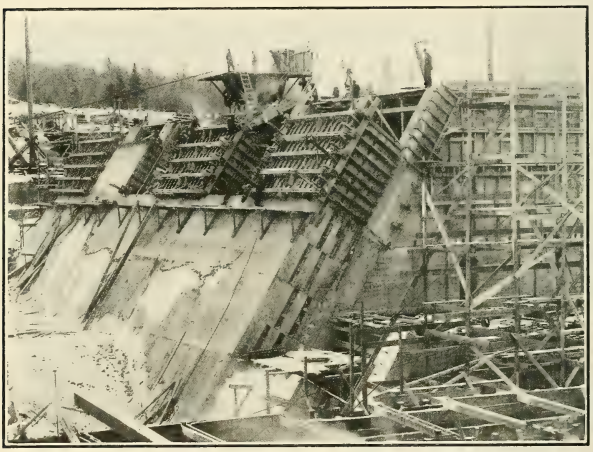

FIG. 19.—BUTTRESSES UNDER CONSTRUCTION, AZISCOHOS DAM.

the freedom from obstructing falsework, and the workmanlike appearance of all parts of the construction, in spite of the fact that this illustration shows the extreme end of the buttressed spillway section at a point when the natural ledge profile occasions irregular form erection to compensate for the abrupt slope of the foundation.

Fig. 19 is taken from a platform central with the river bed looking southwest toward the work completed on the main dam November 9, 1910. In the immediate foreground is seen the framing of the temporary log sluice. This view again demon-

strates the systematic progress and clean finish of the work, which must be credited, in a large measure, to the form construction. It also shows the caulking cleats required on account of the fact that it was necessary to construct the form panels of green boards, resulting in the inevitable shrinkage and opening of joints; the pockets provided in the buttress nosings for receiving the end panel studding of the first cast arch forms; and the texture of the untreated finished concrete surface.

The writer does not intend to convey the idea that sectional

FIG. 20.—GENERAL VIEW OF AZISCOHOS DAM AND SITE.

forms are materially, if any, cheaper per unit of contact surface than the ordinary kind built in place; but their use is strongly advocated on account of the rapidity with which they can be assembled ready for the placing of concrete, without employing an army of carpenters to keep the form work ahead of the concreters. It is undoubtedly true that the unfortunate foundation difficulties encountered in this instance had material effect on the unit cost of the forms on account of concentration and consequent congestion which it occasioned. Had it been possible to spread out the work over a sufficient number of piers, a much more creditable

showing could have been made; as matters stand, the forms have accomplished a saving of from 150,000 to 200,000 ft. board measure of lumber, and have been instrumental in effectively increasing the quantity of concrete placed.

To conclude, attention is called to Fig. 20 which, although of less recent date than some of the former views, conveys a better idea of the magnitude of the structure. The rear spillway piers are completed ready to receive the crest and overfall apron, three of the bays having the forms for the base section of apron in place. In the central distance can be seen the excavated core wall trench and the top of the deep cut at right angles to same for the first buttress of the main dam.

CONCRETE CONSTRUCTION. TESTIMONY OF THE ROMAN FORUM AND SOME MODERN EXAMPLES.

By Alfred Hopkins.*

Some two years ago I had the good fortune to be in Rome at a time when the excavations of the foundations of the Roman Forum were in progress. These excavations gave me an excellent opportunity to study the Roman concrete, and the result of that study I now put before you.

That the Romans were very familiar with concrete and gave careful consideration to its use, examination of the concrete work in the Forum clearly shows. Their concrete contained generally only two kinds of stone—travertine and selcie—in equal parts. Selcie is a hard, closely knit rock, very similar to our blue stone or trap rock in color as well as quality. Travertine is a volcanic rock, not so hard as selcie, considerably lighter in color and was desirable on account of its porosity, which insured a good bond with the cement. In all the Roman work the combination of the travertine and the selcie is clearly distinguishable. The mortar itself was composed of two parts of puzzolan, a splendid natural cement, and one part of lime, made by the burning of marble. In some instances the proportion of lime exceeded this, though this mixture was usually observed and is in general use at the present day. Puzzolan corresponds somewhat to our Rosendale, but is harder, although it has not the strength or tenacity which is so striking a quality in all of the good brands of Portland cement now in general use.

The Romans mixed their concrete exactly as we mix ours—in a general batch, that is, stones, cement and lime were mixed together and then thrown into a wooden form, precisely as we do it to-day. The marks of the wood forms, are at all times discernible, and especially is this so in the corridor of the house of Augustus on the Palatine (Figs. 1 and 2), where the grain of the wood can be clearly seen. These walls are some 24 ft. above the

*Architect, New York, N. Y.

ground level and though the construction of the forms seems to have been carelessly done, as the illustrations show, yet the result is none the less interesting. Here is a splendid opportunity to see concrete and to leisurely inspect it from every point of vantage. Above these concrete foundations rose the Palace of Augustus, formed of those stupendous walls and vaults of brick which here

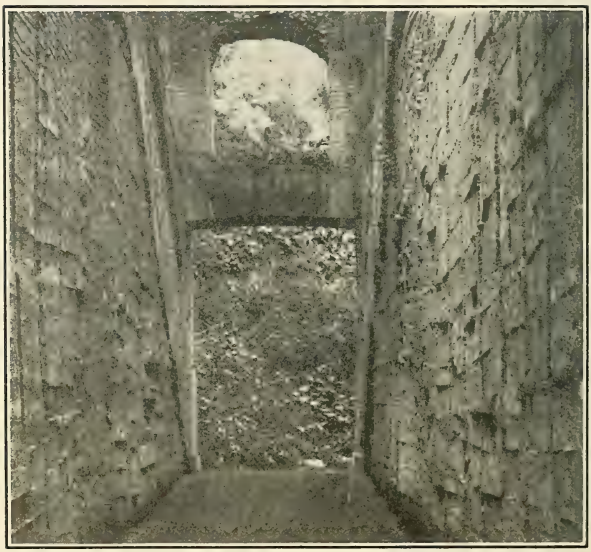

FIG. 1.—CORRIDOR IN THE HOUSE OF AUGUSTUS ON THE PALESTINE HILL, SHOWING MARKS OF WOOD FORMS USED IN THE CONCRETE CONSTRUCTION.

as elsewhere in Rome, thrust their arches through the air with such poise and precision that they are to this day the admiration of every beholder and gave to the Romans their proud position among the master builders of the world. The structure of brick above these concrete walls has succumbed to the ravages of time and to the hand of the destroyer, but the concrete remains without a crack or a fracture that could be discovered by a careful and frequent examination. Its adhesion is perfect, and that there has

not been the slightest disintegration of even the outside surface is attested by the fact that the grain of the wood from the old forms may still be seen on the concrete, though its imprint was made over two thousand years ago.

Some recent excavations at the Arch of Titus (Fig. 3) have disclosed the fact that this structure rests entirely upon a monolithic base of concrete, approximately 45 ft. long, 20 ft. wide and 12 ft. deep. This foundation was poured into wooden frames,

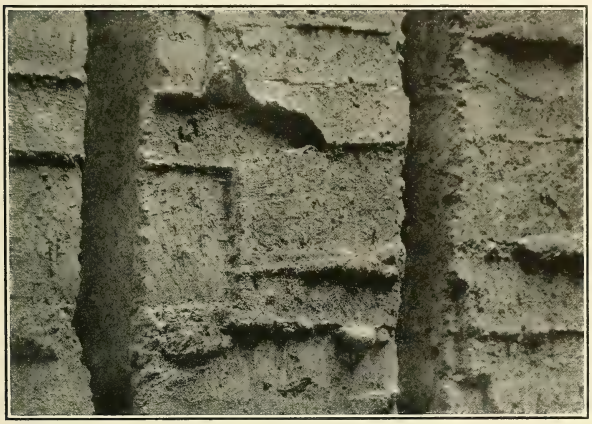

FIG. 2.—HOUSE OF AUGUSTUS. NEAR VIEW OF THE WALLS, SHOWING THE IMPRESSIONS OF THE WOOD FORMS AS WELL AS THE ACTUAL GRAIN OF THE WOOD.

exactly as we should do it now, and when the concrete had set these wooden forms were removed; they were constructed of planks 11 ins. in width with vertical braces 12 by 6 ins., 3 ft. 8 ins. on centers; these braces were put on the inside of the forms, and not on the outside. The excavations at the Arch of Titus, while deep, have very little width, so that it is impossible to get a good photograph of them, but Fig. 4 was taken with the camera pointing directly into the opening. While it gives little idea of the depth, some idea of the surface may be had where the impression of the wooden forms is seen, and the leakage of the concrete

FIG. 3.—ARCH OF TITUS. THE EXCAVATIONS WERE IN PROGRESS ON FEBRUARY 12, 1908, THE DATE OF THIS PHOTOGRAPH. THE PLANKS WHICH GUARD IT SHOW THE EXACT LOCATION.

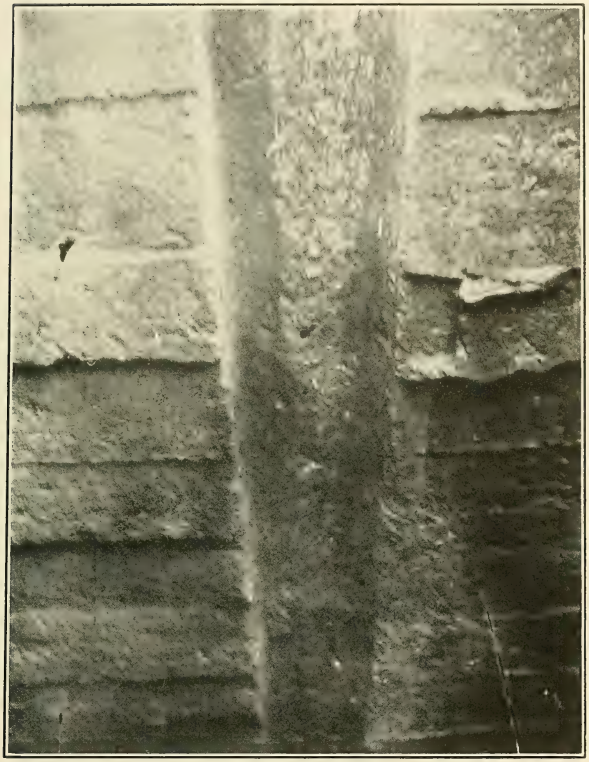

FIG. 4.—ARCH OF TITUS. A NEARER VIEW OF THE FOUNDATIONS. THE MARKS LEFT BY THE WOOD FORMS ARE CLEARLY VISIBLE. THE VERTICAL GROOVE IS THAT LEFT BY THE UPRIGHT BRACE PLACED INSIDE THE FORMS.

between the joints in the planking leaves a slight ridge on the surface, establishing the exact width of the timber used. A very interesting example of the combination of travertine and selcie in the concrete is here found. The selcie and travertine, instead of being mixed together in the usual way were laid in separate layers of all selcie and all travertine; these layers vary little from 7 ins. in thickness and may be easily observed in Fig. 5 where, at the nearest corner directly under the heavy course of travertine rock which forms the base of the Arch, may be seen the first layer of

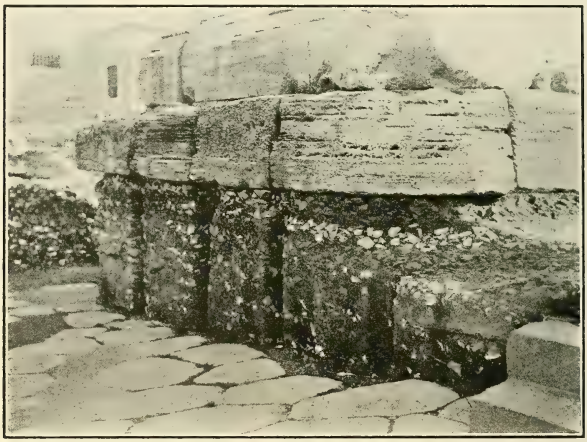

FIG. 5.—ARCH OF TITUS. THE LAYERS OF SELCIE AND TRAVERTINE IN THE CONCRETE ARE CLEARLY SEEN. THE STRATIFICATION AND POROSITY OF THE TRAVERTINE ROCK MAY BE OBSERVED IN THE LARGE COURSE OF TRAVERTINE WHICH FORMS THE BASE OF THE ARCH.

selcie; the selcie is easily distinguished, as it is much darker in color than the course of travertine above. Directly under the first layer of selcie is the first layer of travertine, which, though made up of small stones, may readily be seen to be of the same color as the heavy course of travertine above. Below this first layer of travertine may be seen the second layer of selcie, and below this, in a spot where the traffic has worn it bright, appears the second layer of travertine. These alternations continue with distinctness

FIG. 6.—TEMPLE OF JULIUS. CONCRETE SUBSTRUCTURE.

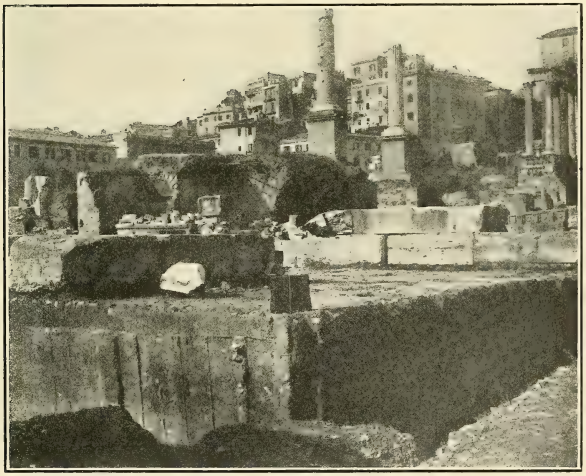

FIG. 7.—TEMPLE OF JULIUS. CONCRETE BASE IN FRONT OF THE RUINS.

in the concrete, but are not quite so apparent in the photograph. This clearly shows how much thought and attention were given to what we are inclined to call unimportant details. After so long a time we can see that the concrete, composed of alternate layers of selcie and travertine, has no especial advantage over a general mix, and that this variant from usual conditions is principally valuable as showing that the subject of concrete excited enough interest, even in those days, to develop experiment, and, perhaps, controversy also.

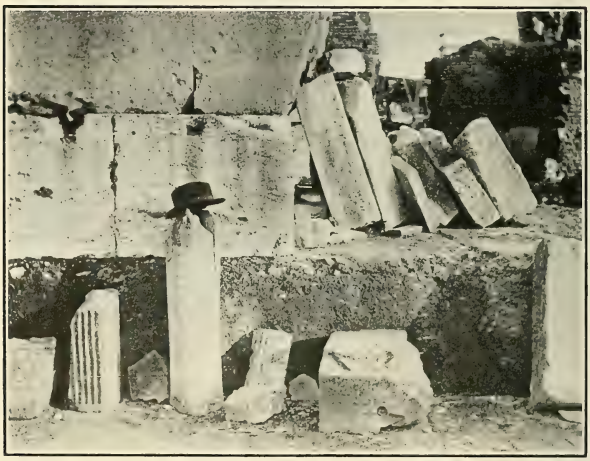

FIG. 8.—TEMPLE OF ANTONIUS AND FAUSTINA. CONCRETE FOUNDATION.

On the east side of the Forum is a mass of concrete which formed the fill of the sub-structure of the Temple of Julius. The general view of this is shown in Fig. 6. Directly in front of the ruins of the Temple of Julius is a large concrete base (Fig. 7) in which also the vertical marks of the wooden forms can be clearly seen. The excavations here do not permit a view of this entire structure, but enough of it appears to give a fair idea of its state of preservation, which is perfect. There is not a crack or fracture in it, and though located in a marshy part of the Forum, it

shows no effect from the moisture to which it has been subject for so many centuries. Some appreciation of its size may be had by noticing in the picture the foot rule which stands in front of the black box, directly on the corner of the monolith.

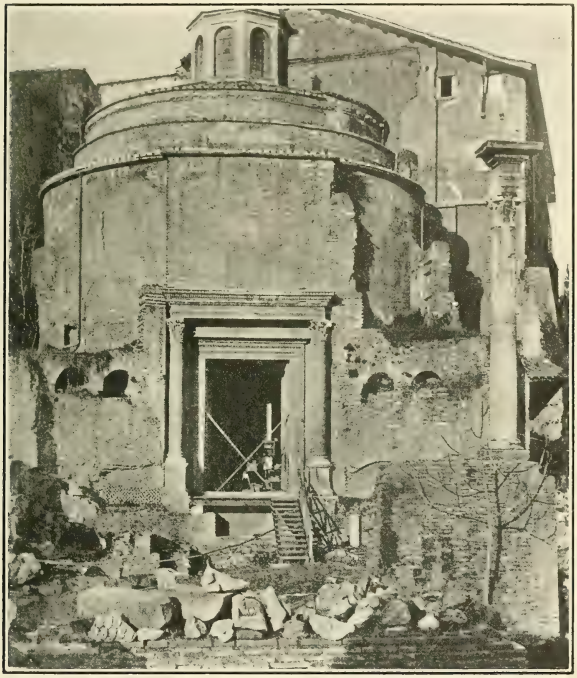

FIG. 9.—TEMPLE OF ROMULUS. GENERAL VIEW.

Back of the Temple of Caesar is the fine Temple of Antonius and Faustina, which also stands on a foundation of concrete, which may be observed in Fig. 8. The hat in the picture shows directly against a block of travertine; this block is the lowest course of the stone structure, and below it may be seen the long, dark monolithic

mass of concrete projecting in front and to the right. This concrete, like the others noted, remains unaffected by every strain that has been put upon it.

Near here the hardest and perhaps the most convincing example of concrete in the Forum is to be seen in the foundations of the Temple of Romulus (Fig. 9). On account of further excavations, it was necessary to cut off the front face of the concrete base, so that the interior structure has been laid bare.

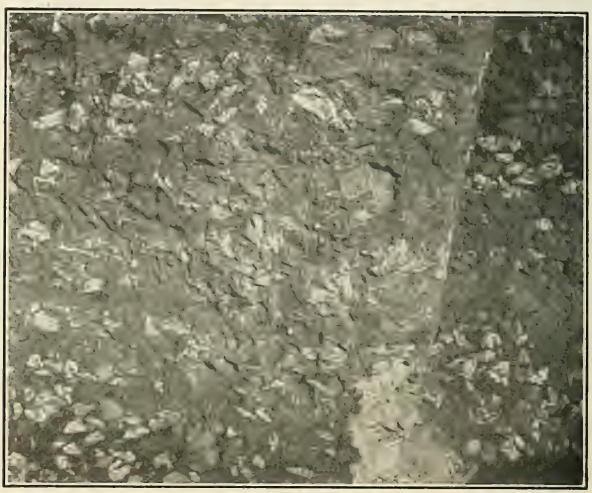

FIG. 10.—TEMPLE OF ROMULUS. PHOTOGRAPH OF CONCRETE, THE FACE OF WHICH HAS RECENTLY BEEN CUT DOWN. THE SPOT, LIGHT IN COLOR, IN THE LOWER PART OF THE VIEW, SHOWS WHERE A PIECE WAS REMOVED.

If anyone could doubt the permanency of concrete while looking at the outer surface, all such doubts must disappear upon a close scrutiny of this inner part. The example given in Fig. 10 was taken with the camera as near to the concrete as it was possible to get it, and the density and compactness of the mass can be appreciated by a little study of the picture. The light color of the travertine fragments shows in contrast to the darker tone of

the selcie. That this concrete has been impervious to any action of the elements is proven by the absolute solidity of the mass. A

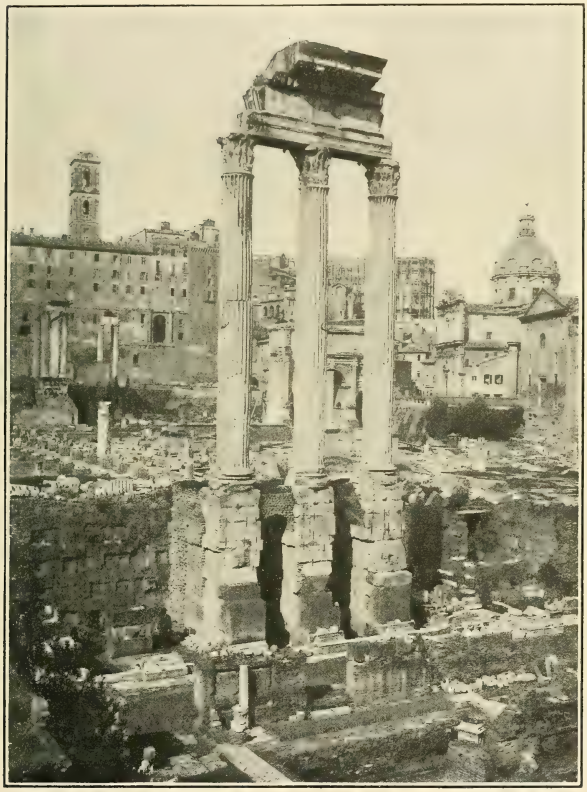

FIG. 11.—RUINS OF TEMPLE OF CASTOR.

piece, the size of a pineapple, was broken off to give to the author, and an examination of the fresh fracture shows that the particles of cement and stone, after having been encased as concrete for

two thousand years, come out with their color as clear and bright as if the puzzolan had just been dug from the Campagna and the selcie and travertine were fresh cut from the quarry. In this example alone we have the most convincing testimony as to the ability of concrete to retain its integrity and to withstand all natural conditions indefinitely.

FIG. 12.—TEMPLE OF CASTOR. NOTE PERFECTION OF JOINTS.

In an article on Foundations of recent date, written by an engineer in one of the weekly engineering journals, it was stated that the Romans used concrete only for their cheaper foundations, and the inference seemed to be that they must have regarded concrete as an inferior building material, and did not employ it in their most important work. This statement may or may not have been a reference to the careful and well-contrived stone sub-

structure which supported the Temple of Castor, frequently, but incorrectly, called Castor and Pollux. These foundations are well known for their size and the care with which they were made. Though the article just referred to was read after my return from the scene, as an architect who is continually confronted with conditions of cost, I took particular interest in the foundations of the Temple of Castor, as the illustrations may show, because of the perfection of their technique—in the main—and what must

FIG. 13.—TEMPLE OF CASTOR. NOTE DOVETAIL OF JOINTS.

have been the great expense required to insure the same. Some idea of their magnitude is seen in Fig. 11, which shows the foundations as well as the three columns on top—all that is now left of the Temple itself. This foundation is some 38 ft. in height, composed of huge blocks of tufa, 2 ft. 6 ins. high, and numbering fifteen courses all told. These blocks of tufa were laid together without mortar, and the joints were so carefully made (Fig. 12) that they must have been rubbed, as the contact between the stones

FIG. 14.—A MODERN REINFORCED CONCRETE RESIDENCE.

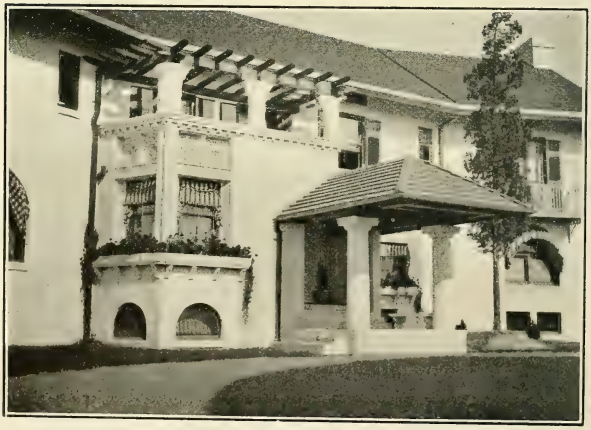

FIG. 15.—METHOD OF ORNAMENTATION OF RESIDENCE.

is perfect at this day. Each stone was dovetailed to the next (see Fig. 13) with painstaking care, and though a better connection would have been made by the use of cement mortar, the architect thought otherwise, and depended on the dovetail to keep his blocks in position. It can now be plainly seen that this was an unwise decision as nothing can be found of these dovetails, though they are supposed to have been of wood; at any rate, they were useless, as the stones have retained their position on account of their great size and weight, and not on account of the connection devised by the builder.

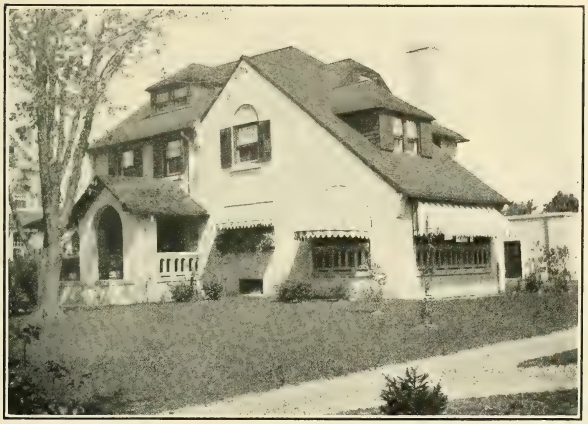

FIG. 16.—A CONCRETE RESIDENCE WITHOUT ORNAMENTATION EXCEPT PORCH RAILINGS.

No one can see the present condition of the foundations under the Arch of Titus (Fig. 4) under the Temple of Romulus (Fig. 10) or the large concrete base shown in Fig. 7, without realizing how immeasurably superior they are to the foundations under the Temple of Castor, just described. It may be that the Romans in their day did not appreciate the full value of concrete, notwithstanding its general and continual use. If they did prefer other methods, but on account of cost or expediency used concrete, then they certainly builded better than they knew; and if there was

any doubt in the minds of the Romans as to the length of life of the concrete, the present condition of the concrete put in by them can leave no possible doubt in our minds as to the value of concrete as an enduring construction. In the few examples here noted it has been shown that it can successfully withstand the most trying test known to the builder—the test of time—and this reaching over no less a span, in human knowledge and experience, than twenty centuries.

I have been asked to give some solutions of concrete construction made by myself and do this reluctantly, feeling that what

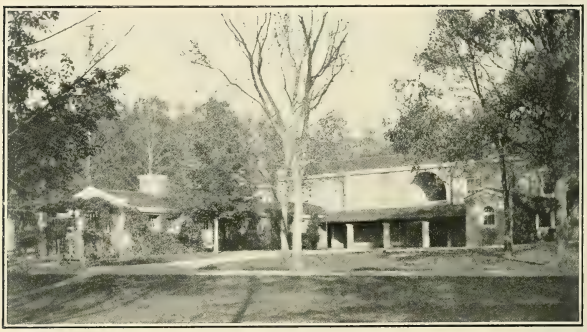

FIG. 17.—REINFORCED CONCRETE FARM BUILDINGS WITHOUT ORNAMENTATION.

any one man may do must be considered insignificant compared with the great works just considered.

I have always regarded concrete as pre-eminently *a construction.* It seems to me a waste of time to try and make actual concrete beautiful; it is a plastic material and while some of the most beautiful things in the world have been accomplished through plastic mediums, yet concrete does not possess the required refinement of texture to mold itself happily into the forms of ornament. It should be regarded entirely as a construction and in that sense it is without an equal. This view may be perhaps set forth a little more clearly in Fig. 14, which shows a reinforced concrete house throughout. There has been no attempt to use the concrete for anything else but construction, but care has been

taken with the window openings and the shapes of these, so that they may be harmonious in the plain wall space. The ornamentation is entirely of colored terra cotta and Fig. 15 will give some idea how the terra cotta has been used. It is to be regretted that the illustrations do not show the color of the terra cotta which was carried out in warm yellows, light greens, and in very small spaces blues and reds.

Fig. 16 shows a small house without ornament whatever except the railing to the porches which, in this instance, has been cast in concrete, but designed in such a manner that it is suitable for its material. All that the architect can hope to do is to have the outline of the house good and the form of the openings agreeable and harmonious. Some farm buildings, Fig. 17, were built in a very picturesque location entirely of reinforced concrete with red tile roofs and where no ornamentation whatever was used, and yet the effect of the different buildings in combination I feel to be reposeful and satisfying.

I am firm in my belief that concrete realizes the ideal as a construction. If we have failures the blame must be laid directly at the door of the individual who by his failure has proved only that he is quite unfamiliar with the material he has essayed to use, and to return to the Roman Forum we find there convincing testimony as to the efficiency and durability of concrete. Looking back through the centuries in which this character has been so notably maintained we must see that here is a material on whose merits we can form a definite and certain judgment, and the judgment thus formed impels us irresistibly to the conclusion that we have no building construction which, viewed from any standpoint, measures up to the incomparable standard accomplished by concrete.

THE USE OF REINFORCED CONCRETE FOR HOSPITALS AND SIMILAR STRUCTURES.

By R. A McColloch.*

As an architectural material concrete is one of the most difficult to handle at once logically, expressively and artistically. Concrete is essentially monolithic, and in its use this character should always be borne in mind and its expression sought in the development of the design. Such a result may probably best be obtained by the use of plain surfaces, not necessarily massive (for the introduction of reinforcement enables the designer to measurably approach the slenderness of the steel framed and enclosed structure), but broken only by surface ornament evidently applied. To so limit the character of the design would appear to restrict the use of concrete mainly to utilitarian structures, but such is not the case. Reinforced concrete may be used as a material expressed in the design for almost every variety of structure except perhaps the purely monumental.

Unadorned, the reinforced concrete structure necessarily expresses the severity of the strictly utilitarian, while the application of surface ornament, especially in color, gives a sense of brightness which may be developed even to the gaiety of the purely ornamental and ephemeral buildings of an exposition, or the light kiosks, pergolas and retreats of a garden or park. Under different development reinforced concrete, adorned with surface ornament, may be used to express the majesty and richness of ecclesiastical design. It does not adapt itself, however, to the usual ornamental forms developed from the use of wood and stone. This view will doubtless be opposed by the majority of those using concrete, as the ease with which the usual architectural forms and details may be formed by molding and casting in cement has tempted many to employ them as an economical method of producing a desired effect. The results, however, are seldom successful as the very

*Chief Engineer, Raymond F. Almirall, New York, N. Y.

method of the production defeats the ends sought. The features, cast all in the same mold, by their very sameness express their cheapness, while the life and expression of carved work, resulting from its sharpness and the inevitable variations of individual pro-

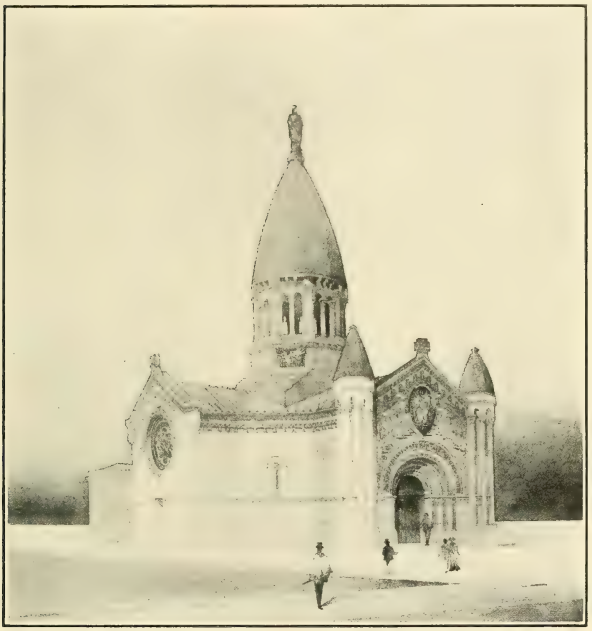

FIG. 1.—SKETCH FOR MORTUARY CHAPEL FOR CALVARY CEMETERY, LONG ISLAND CITY, N. Y.

Raymond F. Almirall, Architect; P. J. Carlin Construction Company, Builders.

duction, are totally lost. That this same argument may be used against other molded work is true, but in the first place the conditions are not exactly the same, and in the second place we all do many things, to obtain economical results, which are opposed to architectural excellence and to good taste.

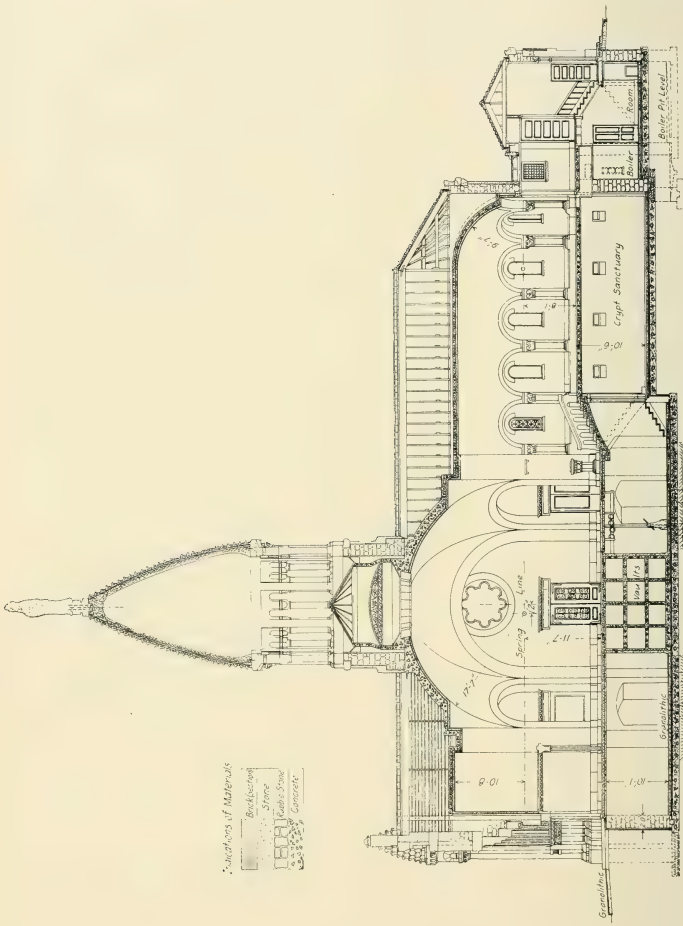

FIG. 2.—SECTIONAL VIEW OF MORTUARY CHAPEL FOR CALVARY CEMETERY, LONG ISLAND CITY, N. Y.

The materials most in use for molded architectural forms are terra cotta and plaster, the latter used only in interior work. Terra cotta differs from concrete in that the process of its manufacture, by molding and burning, while it causes the loss of the clearness and sharpness of carved work, produces the irregularities in the material which are essential to the life of the design, by banishing monotony. It is doubtful if any two pieces of terra cotta from the same mold are ever exactly alike. In plaster work, it is true, exact duplication is obtained, but plaster ornament lends itself to a sharpness, clearness and richness of detail impossible to obtain in concrete. Moreover, its use in interior work, with small motives and multitudinous repetitions, gives by the very volume of these a sense of richness and magnificence appropriate to its position.

There are two other architectural uses of reinforced concrete as a material of construction: first, its use in connection with other materials in a compound structure, where the design expresses combination, concrete, metal and wood being severally used where their employment is appropriate; and second, its use purely as a structural material, usually as a substitute for steel, no expression being given to the material in the design, which may employ any of the usual materials for exterior and interior finish.

As an example of the last use may be taken the Mortuary Chapel at Calvary Cemetery, Long Island City, built in 1905. A sketch of the same is shown in Fig. 1, and a cross section in Fig. 2. This church is an octagonal cloistered vault, with short nave and transepts and a deep sanctuary, all barrel vaults, with a spherical apse. Over the center octagon is a large lantern. The walls are of rubble stone, faced with limestone ashlar molded and carved. The main roofs are tile, the lantern limestone with a high pointed dome tiled on the outside, and surmounted by a heroic statue. The entire interior walls and vaulting are in face brick, backed with and carried by reinforced concrete. The lantern, which weighs over 400 tons, is carried on four intersecting arches, resting on eight masonry piers. The arches also carry the cloistered vaulting, which with the lantern weighs about 800 tons. Between the arches the vaulting is reinforced, and in addition a heavy iron ring is placed at the lantern opening, and one above the

tops of the large arches giving on the nave, transepts and sanctuary, to take the horizontal vault stresses. The barrel vaults are also reinforced. All the reinforcement is run well down into the masonry piers and walls.

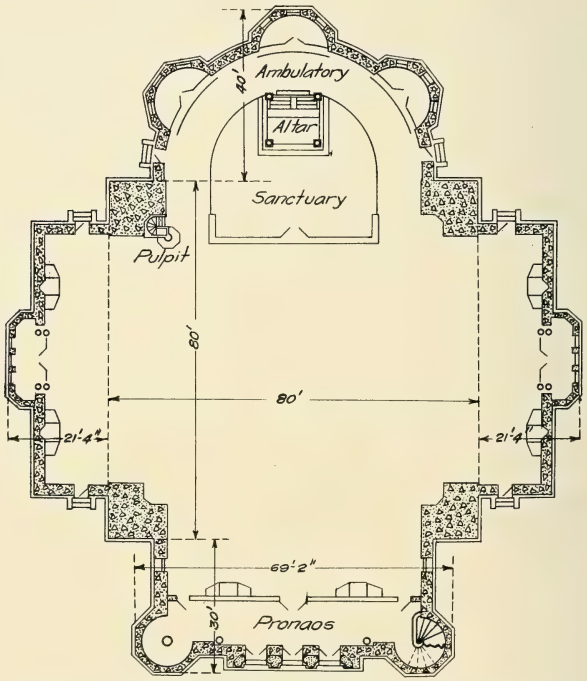

FIG. 3.—PLAN OF HOLY TRINITY CATHEDRAL AT KINGSTON, JAMAICA.
Raymond F. Almirall, Architect; Walker-Fyshe Company, Builders.

The special advantage of reinforced concrete in this work was the simplicity of construction possible by its use. The brick lining, only 4 ins. thick, was laid on carefully and accurately erected false work, lined with slat forms, and frequent anchors were inserted and built in the joints. The reinforcement was then placed

and the concrete poured, carrying up the rough exterior forms as the material was placed. The brick lining and structural work were thus made practically homogeneous, a result which could not be obtained with any other materials. The floors, stairs and burial vaults are also of reinforced concrete, used on account of the small space occupied in proportion to the strength. The burial vaults are in the center of the crypt, and the floor is a flat slab resting on the walls and on the central vault, which latter is divided into receiving spaces for the coffins.

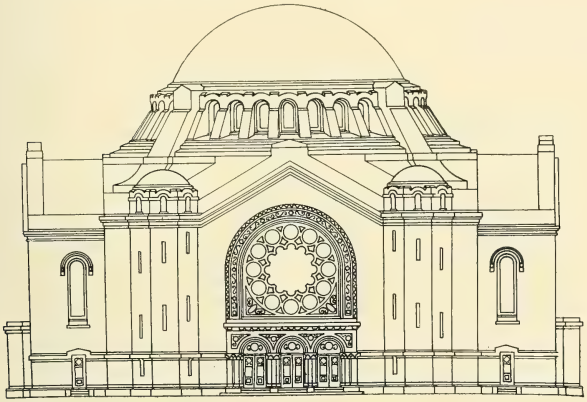

FIG. 4.—FRONT ELEVATION OF HOLY TRINITY CATHEDRAL AT KINGSTON, JAMAICA.

As an example of the possibilities of architectural expression in reinforced concrete may be cited Holy Trinity Cathedral, at Kingston, Jamaica, erected after the recent earthquake. This church, Figs. 3, 4 and 5, is similar to the chapel just described, but is entirely finished in concrete, both inside and out, with applied ornament only. On account of the danger from earthquakes, this building was designed in the form of a box, and can be practically stood on one corner without injury. Not only is the dome, which is carried on four intersecting reinforced arches, entirely reinforced, but all the walls are reinforced in two directions, the piers

are reinforced concrete, the foundations are reinforced, and the pier foundations are tied together by heavy reinforced concrete beams run from corner to corner and also diagonally. This is probably the first building in which the feature has been employed of tying together all the foundations by long diagonal and horizontal reinforcement bars extending across the building from one footing to another, and used not to distribute load, but purely to insure against relative lateral displacement.

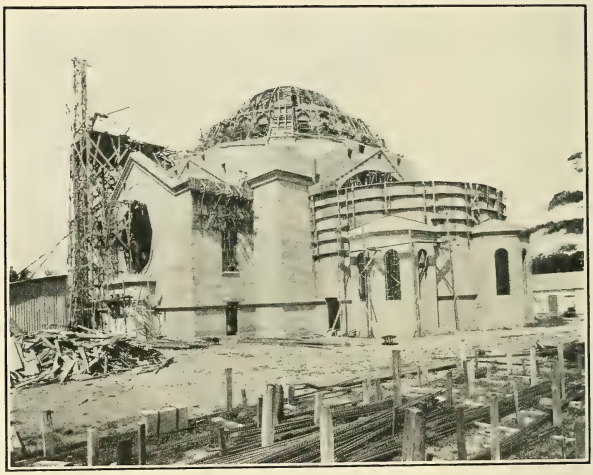

FIG. 5.—HOLY TRINITY CATHEDRAL, KINGSTON, JAMAICA, UNDER CONSTRUCTION.

While the above examples illustrate the advantage of reinforced concrete for the solution of special problems not easily, if at all, attainable by other means, its chief advantages are to be found in its use in every-day work, especially in buildings of an institutional character, such as hospitals, schools and homes for the care and treatment of numbers of persons. As an example of such work may be taken the Sea View Hospital now under construction on Staten Island. A view of the proposed structures is

shown in Fig. 6. A more detailed examination of this group of buildings will illustrate both the special advantages of reinforced concrete, and also some points which should be guarded against in its use. The hospital consists of eight ward buildings, a dining hall and kitchen building, an administration building, a surgical building, a power house and laundry building and an ambulance house. It is proposed also, at some future time, to erect a pathological building, a hydrotherapeutic pavilion, a staff house, a Nurses' home, a building for the help, and a chapel, to complete the institution.

All these buildings belong to the class designated as compound structures, reinforced concrete, structural steel and iron work, terra cotta, tile and sheet metal being used in the same building, where and as appear appropriate. The ward buildings, Figs. 7, 8 and 9, which are four-story and basement structures, are constructed of reinforced concrete walls, floors and roofs, the exterior concrete walls being exposed in finish, and decorated by colored faience in a frieze and band courses. The roofs are tile; the superstructures are steel covered with sheet metal. The verandas have iron columns and railings, and reinforced concrete floors. The foundation walls are of reinforced concrete, as are also the main stairways. The design, as may be seen, is very open, the percentage of horizontal wall openings being about 60, and for this the reinforced concrete, combining the properties of steel and masonry in one material, was especially advantageous. This very slenderness entailed, however, certain obstacles which proved more serious than previously contemplated. On account of the number and variety of openings and slenderness of the piers the form work was expensive, running at times as high as 25 cts. a face foot, while the accurate placing of steel and pouring of the vertical members precluded rapid progress. Also, in spite of all possible care, the vertical and horizontal arrises of the openings were in many cases not true, the lintels especially showing sag at the center. While this defect would be entirely immaterial in a purely utilitarian structure, such as a warehouse, when occurring in an architectural design its effect is too noticeable to neglect, and its rectification has been slow and expensive.

The buildings were designed with a band course at each floor,

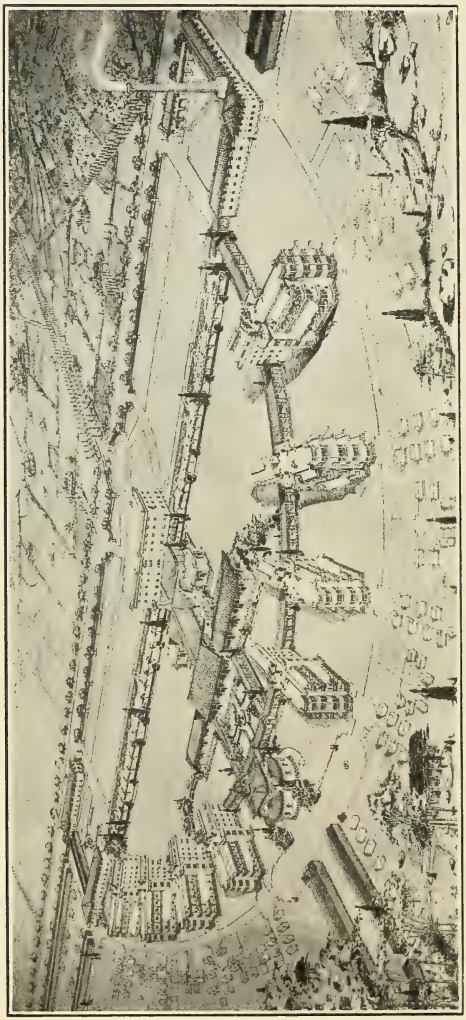

FIG. 6.—GENERAL VIEW OF PROPOSED SEA VIEW HOSPITAL BUILDINGS ON STATEN ISLAND, N. Y.
Raymond F. Almirall, Architect; L. A. Burke and Sons Company, Builders.

the intent being that the concrete should be placed a floor height at a time, the joints between day's work occurring at the band courses. In the attempt to attain this under the conditions the form work was in many cases not made sufficiently true and smooth, and consequently the finish as left by the forms was too rough to remain. Foreseeing this possibility the specification provided that the finished surfaces should be rubbed with carborundum brick or otherwise satisfactorily treated to give an even homogeneous surface free of form marks and lines. After considerable experiment a finish was adopted made by trowelling on a thin coat of cement

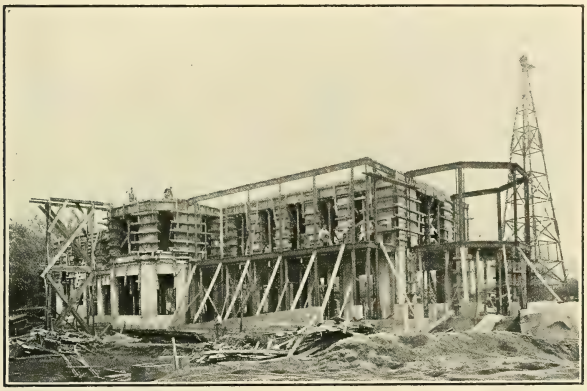

FIG. 7.—WARD BUILDING NO. 5, UNDER CONSTRUCTION, SEA VIEW HOSPITAL.

over the surface after thoroughly wetting, and then immediately brushing down with a whitewash brush. This finish has now been on some 6 to 8 months and is satisfactory in appearance and durability.

The interior structural work consists of reinforced concrete columns, beams and girders with flat floor arches, giving an absolutely fireproof structure. Some difficulty has been encountered, however, as the unavoidable size of the columns and girders interferes with the proper running of piping and conduits causing awkward spots to finish properly in a number of places in the

interior. In order to obtain flat ceilings it was necessary to wire lath them throughout the interior, though the veranda ceilings are left exposed and finished in a manner similar to that of the outside walls.

The exterior decoration of the concrete structure is obtained chiefly by the use of faience, a large frieze occurring at the top floor, and a band course at each story. The columns and rails of the verandas are made of iron, to gain lightness with strength in a

FIG. 8.—WARD BUILDING NO. 6 UNDER CONSTRUCTION, SHOWING FORMS FOR CORNICE. SEA VIEW HOSPITAL.

more marked degree than in the body of the structure, thus separating the building itself from the subservient additions and giving each an expression proper to its purpose. Similarly the superstructures are finished in sheet metal, giving a lightness of design proper to their secondary nature, and adding a decorative feature to the top of the building. Except for the irregularities in finish referred to, due to improper workmanship, the results have been exceedingly satisfactory. The use of a band of applied ornament at each story to cover the joint between the successive stages

of erection has been especially successful, and has obviated much of the streakiness due to the efflorescence occurring always in a marked degree at a joint where fresh concrete is applied to that already set.

In the dining hall, surgical pavilion, administration building and power house, laundry and ambulance house a different construction has been adopted. On account of the cost of the form work and slow progress of the reinforced concrete walls the build-

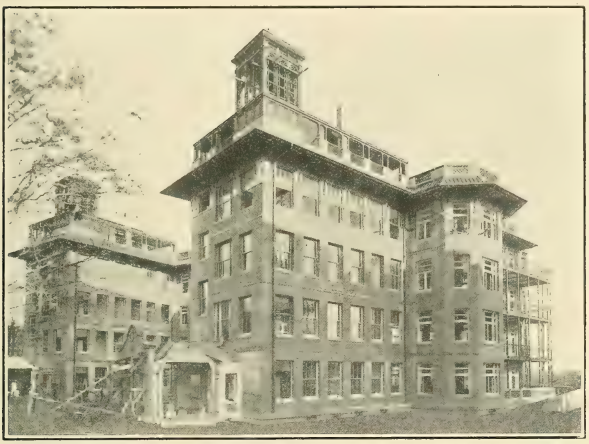

FIG. 9.—WARD BUILDING NO. 3, SHOWING COMPLETED CONCRETE WORK AND ORNAMENTATION IN PLACE. SEA VIEW HOSPITAL.

ings have been designed with reinforced concrete and steel skeleton frames and floors, and enclosed with special terra cotta block walls, finished outside in stucco to match the finish of the ward buildings.

The footings are reinforced concrete, designed in the usual manner, except that the foundation walls have been reinforced to act as girders and distribute the column loads along the wall footings. This provision was necessary on account of the light weight of the independent terra cotta enclosure walls. To reduce the unit

stress under the column footings to that of the wall footings would have entailed excessively large foundations, and the load was accordingly distributed by designing each footing of fixed length, center to center between columns, and fixed width determined by the wall width. The requisite variation in size and shape to give the proper unit pressure and to place the center of pressure below the center loading was obtained by adding a rectangular block of proper size, shape and position, making a T footing. This block is determined by the following formulæ:

$$X = \frac{2A\ (c-b)+ba}{a_2}$$

$$Y = \frac{a_2^2}{2A\ (c-b)+ba_1}$$

In which b is the fixed width and l the fixed length of the footing, Y the unknown width and X the unknown length of the added block, A the total footing area, a the area of bl, and a_2 the area of XY, the areas being known quantities determined by the loading, and c is the distance of the center of pressure from the outer edge of the footing *i. e.*, from the top of the T. The position of XY is such that the center of gravity of the two rectangles falls on the center of pressure.

Similarly in the case of a corner column, where there are two known rectangles and a third to be determined the formulæ are:

$$X = \frac{Ac_2 - a_1\frac{l_1}{2} - a_2\frac{b_2}{2}}{a_3}$$

$$Y = \frac{Ac_1 - a_1\frac{b_1}{2} - a_2\left(\frac{l_2}{2}+b_1\right)}{a_3}$$

In which b_1l_1 and b_2l_2 are the known rectangles whose areas are a_1 and a_2, respectively, X and Y are the co-ordinates of the center of gravity of the third rectangle, whose area is the known quantity a_3; c_1 and c_2 are the co-ordinates of the center of pressure, c_1 being parallel to Y and c_2 to X, and A is the total area.

The unit pressure, which may be assumed, is, of course, limited by the conditions of the design, and trial footings must be laid out, that the T footing may be obtained in all cases and not be

irrational. For the corner footings the result is usually a compromise, as the added area to comply strictly with the conditions of coincidence of center of gravity and center of pressure will generally occur as an isolated block or as partly within the other rectangles; it should be so located that the overbalance of pressure will tend to throw the corner inward.

The columns are built up of four angles latticed, and are filled with concrete which also completely encases the steel work with not less than 2 ins. of material. The column is so designed that considered as a steel column neglecting the concrete, it will have such proportionate length to radius of gyration that the allowable stress in the steel under the column formula adopted will be about 12,000 lbs. per sq. in. So designed the column is of sufficient strength to carry the entire dead load. The concrete inside of the steel angles is figured to carry the live load at a ratio of 15 to 1, giving a unit stress of about 800 lbs. per sq. in., and ensuring the equal compression of the two materials. Should the live load be such that the concrete cannot carry all of it, the amount of steel in the angles is increased to care for the additional loading at the unit stress assumed. While a column of this design will undoubtedly carry with safety more load than assigned by the above method of calculation, both because the steel column is stiffened by the addition of the concrete, and because the concrete is so completely restrained that it will safely carry a heavy load, in view of the lack of absolute knowledge of the properties of such columns, in the absence of tests, it is undoubtedly better to use a conservative basis of design.

The special advantage of reinforced concrete is evident in this column and footing design. The wall columns extend into the concrete foundation walls, which are of such height that the adhesion between the steel angles of the column and the concrete wall takes up and distributes all the column load into the wall, the column requiring no base. The concrete foundation walls have been reinforced with rows of rods about mid-way of their height. While this appears opposed to every principle of reinforced concrete design, in reality it is the most economical position. On account of the conditions of the design—the basement floor, in most cases being 5 ft. or more below grade, and the footings in all

cases being at least 4 ft. below grade to avoid frost—the concrete wall is never less than 4 ft. deep and usually 6 to 8 ft. Further, from 4 to 6 ft. penetration into the concrete wall was necessary to give sufficient area for adhesion to transfer the load from the steel angles of the columns to the wall. The wall being continuous, when taken as a girder must act as a continuous one. At the columns the upper half of the wall is figured as a cantilever, while in the intervening portion the lower half is figured as a simply loaded beam, resting on the cantilevers, the hinge point being assumed at a point giving equal reinforcement to cantilever and central beam. This assumption, which is justifiable on the ground that with deflections the actual hinge point will shift until the stresses are equally distributed, enables the pouring of the entire wall in two operations and with only one set of bars to be placed, thus gaining both economy and rapidity of construction.

The reinforced concrete columns from the manner of design have a length of about 20 diameters, giving a column much more slender and occupying much less floor space than the usual form of reinforced concrete column, and requiring no additional covering for fireproofing. As the column is open on four sides the rods of the reinforced concrete girders may be run through, forming an absolute bond, while the column being a stable structural steel member affords simple strong connections for steel girders.

The floors are of terra cotta block and reinforced concrete beams, with a continuous slab above of such thickness that the neutral axis of the beam falls within or at the lower surface of the slab. While this design gives a heavier floor than one of flat slabs and widely spaced beams, it dispenses with all ceiling furring, giving a flat ceiling throughout, and permitting of the placing of partitions irrespective of the structural design and without resort to metal furring and lath. To further obtain this result, where heavy girders were required which could not economically be designed within the depth of the floor system, steel beam girders have been employed, using in many cases the Bethlehem girder beams, which give the greatest strength in proportion to depth of any rolled sections on the market.

In closing, let it be said that this paper is not presented to further any particular theory or form of design, but merely to

explain some of the conditions under which reinforced concrete has been found at once the most practical and most economical material for use in various structures, to meet the conditions governing in general architectural practice, and to combine the requirements of good construction and design at the lowest practicable cost.

REINFORCED CONCRETE SCHOOLS

By John T. Simpson.*

On the 4th day of March, 1908, there was flashed over the telegraph wires to every newspaper throughout the length and breadth of this land, the news of a great disaster. In a schoolhouse at Collingwood, Ohio, a suburb of Cleveland, one hundred and sixty-five children lost their lives through the breaking out of a fire during school hours.

The building, known as Lake View School, was of typical schoolhouse construction,—brick walls with wooden floors, partitions, stairs and corridors. The entrance doors to the building, hinged to swing into the corridor instead of out, were largely responsible for the large loss of life. These doors were closed and the children crowding behind, made it impossible for them to be opened. The usual so-called fire protection, in providing iron stairs or fire escapes, was installed in the building, but seemed to be of no avail to prevent loss of life. The conflagration was supposed to have started by the overheating of the boiler plant, which set fire to the wood floors directly over and which was quickly communicated to the building, and, despite the efforts of the people attracted by the fire, they were compelled to stand helplessly by and see the pupils burnt to death. Fig. 1 shows the building as it appeared after the fire.

This frightful loss of life, led the writer, together with his associates, to take up seriously the investigation of fireproof schools of reinforced concrete. Prior to this, endeavors had been made to interest various Boards of Education in the fireproofing of brick buildings. It was found, however, that the architects would often rather cut out the fireproof construction, if the cost exceeded the appropriation, than change the architectural features of the building to bring the cost down and with the old style of steel beams and fireproof arches, it was generally found impossible to bring the prices within the amount available.

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A few weeks after the Collingwood fire, the Board of Education of Irvington, N. J., brought out plans for the erection of four class room building, and upon presenting the matter to them, they consented to have an alternate proposition submitted covering a reinforced concrete building. The original design called for typical brick walls with wood floor construction. When the bids were received, it was found that a reinforced concrete school could be

FIG. 1.—LAKE VIEW SCHOOL, COLLINGWOOD, OHIO, WHERE 165 CHILDREN LOST THEIR LIVES.

built for $300 less than the best prices received on the basis of brick and wood, and, as a result, to the town of Irvington, N. J., must be given the credit of being the first to adopt this type of building for public schools. After two years of service, the building (Fig. 2) has proven so satisfactory that they have adopted this method of construction for all new schoolhouses.

While this building was in progress of construction, the Board of Education at Summit, N. J., who were planning to erect a nine class room and assembly room building, visited the work and were

so well pleased with the construction, that they adopted reinforced concrete for their new Lincoln School (Fig. 3). A typical class room of this school is shown in Fig. 4, and Fig. 5 illustrates the kindergarten room.

The neighboring town of Chatham, N. J., a few months later obtained bids on both brick and wood and reinforced concrete. The result of the bidding showed that a reinforced concrete building could be built for the same price as the building of brick and wood.

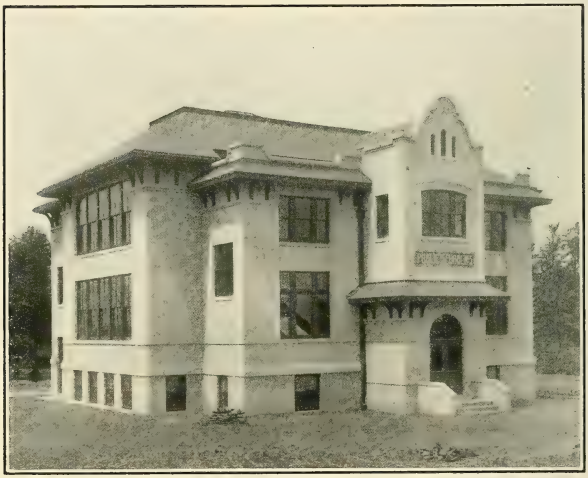

FIG. 2.—PUBLIC SCHOOL NO. 5, IRVINGTON, N. J. JOS. B. ALLEN, ARCHITECT.

Unfortunately, however, as is often the case, the appropriation was made before the plans were drawn and as the bids for a brick and wood building were taken on separate items, the Board was able to contract for as much of the building as the appropriation would provide for, and later made another appropriation to finish the work. This building (Fig. 6) is an exact duplicate, in floor plan, of the building adopted about the same time by the Board of Education at Madison, N. J., for their Central Avenue School, and which building was built in reinforced concrete (Fig. 7). A view of the

main entrance is shown in Fig. 8, illustrating the good, clean cut detail, particularly the lettering, that can be cast in concrete, without going to unusual expense when treated in simple lines.

It should be noted that in the Chatham Building, the second story walls are but 8 ins. thick, the cornices were made of wood, the flashings of tin and the ceilings of stamped metal in order to keep the cost as low as possible; but notwithstanding this, the price for

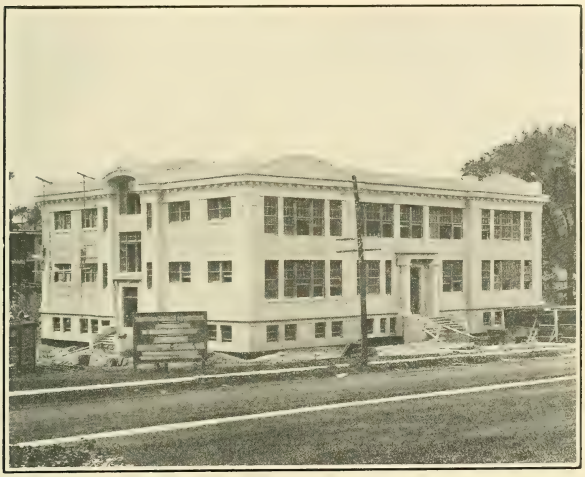

Fig. 3.—Lincoln Public School, Summit, N. J. H. P. Alan Montgomery, Architect.

the reinforced concrete building was the same as that paid for the brick and wood structure.

At Millburn, N. J., competitive bids were taken on brick and wood and reinforced concrete on a four class room building. The average bid on the brick and wood basis was $5,000 higher than the price on the reinforced concrete basis. One bid, however, on the brick and wood was about the same as the price on the concrete basis. The Board decided in favor of the reinforced concrete building (Fig. 9).

Perhaps the best example of what can be done in reinforced concrete was the result of the bidding on Public School No. 11 at Bayonne, N. J. (Fig. 10), and as this building is larger, the comparison would necessarily be better. This building contains twenty-seven class rooms, teachers', principal's rooms, library and an assembly hall seating 1,000 people. This does not include any of the rooms in the basement, several of which are used for class purposes.

The lowest bid received on the basis of brick and wood was

FIG. 4.—TYPICAL CLASS ROOM, LINCOLN PUBLIC SCHOOL, SUMMIT, N. J.

$132,700. The contract for the construction of the building was awarded on the reinforced concrete basis for $111,000,—a saving of $21,700. All bids were without heating and plumbing work.

Recently the Board of Education at Nutley, N. J., visited the Central Avenue School at Madison, and after making an exhaustive investigation, comparing the cost of this building with what they had formerly paid for brick and wood, adopted the plans of this building, and contracts have been awarded to duplicate this school at Nutley.

In the design of a reinforced concrete school building, there are many short-cuts to economy which are not possible with any other material and in all the schools above referred to, no changes have been made in the various items of finish, trim, painting, lighting, plumbing or heating work to obtain these low prices. In every case of competition, the comparison has been fair. The specifications drawn for the original building so far as they would apply to a fireproof building, have been followed strictly. Only

FIG. 5.—KINDERGARTEN ROOM, LINCOLN PUBLIC SCHOOL, SUMMIT, N. J.

the best grades of materials and workmanship have been used throughout the construction.

One of the first items of saving was found to be the area of ground covered by a concrete building, being considerably less than that occupied by one of brick construction, for the reason that the corridor partitions and division partitions between class rooms, together with vent stacks, etc., were usually built of brick, being in size anywhere from 12 to 20 ins. in thickness. As 4-in. partitions take their place in concrete construction, a considerable amount of space was gained. This amounted to 594 sq. ft. on the ground

area between the Chatham and the Madison buildings, with all class rooms the same size.

The practice of the writer has been to use a 10-in. thickness for outside concrete walls with a furring of 2-in. hollow blocks. This has given very satisfactory results and in no case has there been any discoloration of plastering on the interior of the building due to dampness. This is also largely due to the fact that the exterior of the buildings are treated with a liquid concrete; a number of excellent materials being now available for this purpose.

It was also found in designing school buildings in reinforced

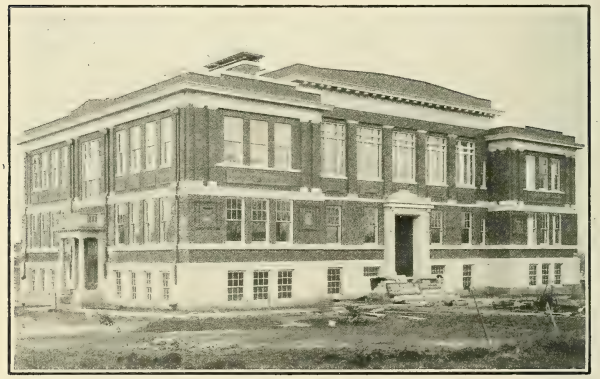

FIG. 6.—PURLIC SCHOOL, CHATHAM, N. J. SAME FLOOR PLAN AS CENTRAL AVENUE SCHOOL, MADISON, N. J., BUT CONSTRUCTED OF BRICK AND WOOD AT NO REDUCTION IN COST. R. S. SHAPTER, ARCHITECT.

concrete that a great deal of floor space could be saved owing to the simplified method of construction, it being possible to disregard the question of supporting walls and by using the column and beam construction to so arrange the beams as to give a pleasing panel effect in the class rooms.

Another item that enters into the cheapening of the building is the fact that it is unnecessary to do the usual plastering on the concrete surfaces. For basement walls and ceilings, the work is left practically as it comes from the forms, touching up only such spots

as may be found necessary, afterwards finishing with cold water paint or liquid concrete.

In the class rooms and other rooms throughout the building, the concrete surfaces are finished under a wood float, using Portland cement and fine sand. Where care has been exercised in the construction of the forms and placing of the concrete, this is found sufficient to turn out an excellent finish. The furring of the side walls and partitions are covered with two coats of plastering.

One of the problems which has not yet been satisfactorily solved is the finish of the floors of class rooms. While the use of

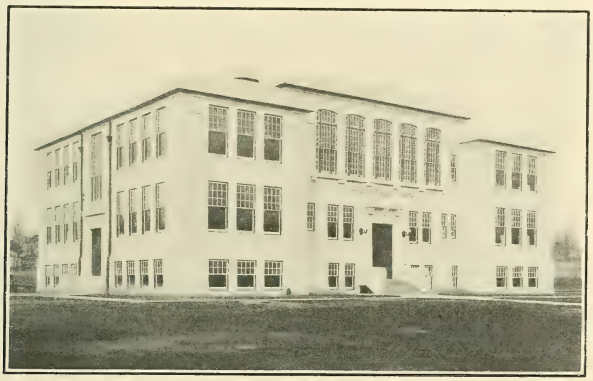

FIG. 7.—CENTRAL AVENUE SCHOOL, MADISON, N. J. H. K. CONKLIN, ARCHITECT.

cement finish for corridors, basements, stairs, coat rooms, toilets, etc., has proven satisfactory after being treated with a liquid concrete to prevent dusting up, the same construction has not been adopted for class rooms. Most boards object to the children sitting with their feet on the concrete.

It has been the practice of the writer to economize in the floor construction by laying 1 in. of sand upon the top of the finished concrete slabs, placing on this diagonally the form lumber used in the construction of the building, nailing together at the edges and joints, and placing rosin sized building paper and covering with 7/8-in. wood flooring, usually of maple or comb-grained yellow pine.

As the use of wood floor carries with it the use of wooden base, this is not an ideal finish for a school room and the writer is now endeavoring to have adopted in some future school work, cork carpet laid directly on the concrete slabs. This makes it possible to use a cement base with sanitary cove around all walls, also the added advantage of a noiseless and warm floor, and from estimates made, floor covering of this kind can be installed at about the same cost as the present method of wood finish.

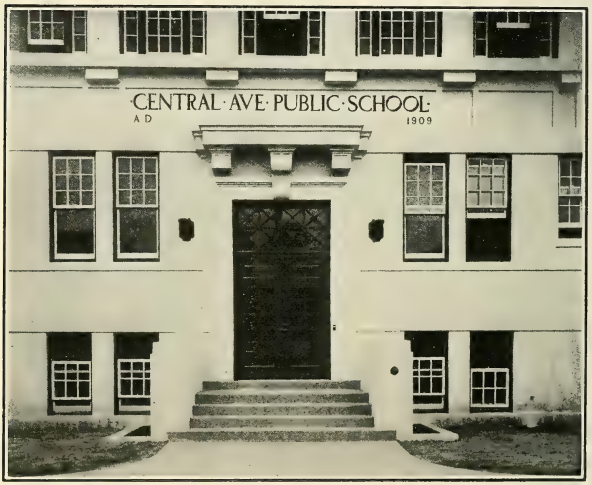

FIG. 8.—MAIN ENTRANCE TO CENTRAL AVENUE SCHOOL, MADISON, N. J.

As the pupils bring in more or less dirt on their feet, a greater portion of which is left on the stairs, keeping the stairs clean becomes an important matter. The former method of applying safety treads to the top of the treads with grooves between the rows of lead has been supplanted by the use of a sanitary tread (Fig. 11), designed by the writer, and which is set into the cement at the time the tread is finished, providing a surface level on top and permitting the stairs to be easily cleaned. This tread furnishes protection against slipping on the edge of the step and is especially

adapted to subways or other places where traffic is heavy and more or less continuous.

The present practice is to use wooden windows and doors in school buildings, and as those made of sheet metal are too expensive, and steel sash have not proven sufficiently tight against drafts to warrant their use for this type of building, the writer is working on designs for hollow steel sash and hollow steel doors, which it is expected can be supplied at about the same price as is now paid for these in wood. In the use of steel windows and doors, trim around

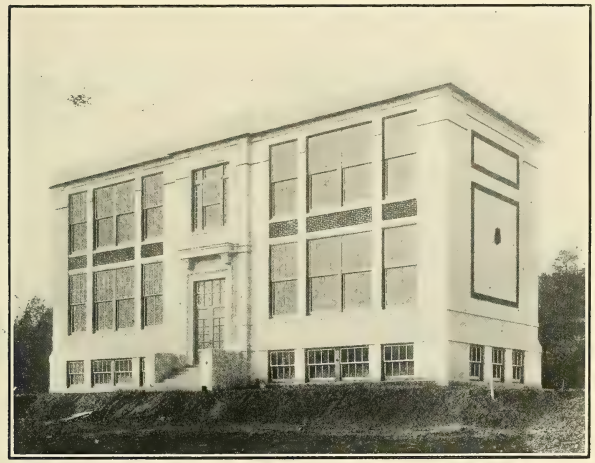

FIG. 9.—WYOMING SCHOOL NO 3, MILLBURN, N. J. R. C. HUTCHINSON, ARCHITECT.

the same will be entirely eliminated, thus doing away with all places where dust is liable to accumulate, and at the same time providing an absolutely fireproof construction. The blackboards also would be set in metal frames.

The installation of a vacuum system of cleaning for removing dust from the school building, is not an expensive item, costing about $50.00 per class room, and should be part of the equipment in all school buildings; the pipes can be connected to the chalk troughs

under the blackboards for the removal of all chalk dust. Where electric current is not supplied to a building, the system can be successfully operated by water motors.

The proper heating and ventilating of a school building is perhaps one of the most important items in the construction. A number of systems have been devised and recommended for this class of work. The use of hot air heating with mechanical ventilation is still found to be satisfactory, though some boards prefer the use of steam or hot water under thermostatic control.

During the cold weather of January, 1910, every schoolhouse in Irvington and Summit, N. J., with the exception of School No.

FIG. 10.—PUBLIC SCHOOL NO. 11, BAYONNE, N. J. R. C. HUTCHINSON, ARCHITECT.

5 and the Lincoln School, were closed for a certain length of time on account of the inability of the heating systems to make the rooms comfortable. In these two buildings, built of reinforced concrete, sessions were held for the full day with all the pupils as comfortable as usual. This demonstrates in a practical manner, that concrete buildings, being more dense than those of brick, are more easily heated.

In architectural appearances, buildings of reinforced concrete can be made more beautiful than by the use of any other material within anything like the same cost. The architect in handling this material, can readily obtain at very low cost, good and expressive details, but the imitation of brick and stone work should not be con-

sidered, as the joints used in brick and stone work are more or less a part of the design, which in concrete, of course, do not exist.

Concrete being of one color, makes it necessary to design the building in mass rather than color. The use of decorative tile and tapestry brick can be very successfully employed for ornamentation, though a too liberal use of the same is not to be recommended.

There is no material perhaps which adapts itself so readily to rapid construction as reinforced concrete. In many cases, when used for suburban school construction, the entire amount of material such as sand and gravel, can be found at the site of the building, making a still further reduction in the cost. There is no waiting for

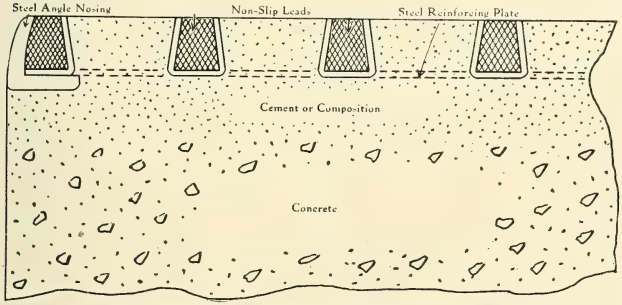

FIG. 11.—CROSS-SECTION OF NEW DESIGN OF SANITARY SAFETY TREAD, USED ON STAIRWAYS OF SCHOOLS ILLUSTRATED.

materials in reinforced concrete work when properly handled. Good reinforcing bars are on the market and readily obtainable from stock. Broken stone, gravel and sand can usually be delivered to the building site within a day or two after the signing of the contract, and there is no delay for iron lintels, stone or terra cotta trimmings, thus permitting the building to be pushed to a rapid completion.

With the improvement in the construction of reinforced concrete buildings, the wheelbarrow is being eliminated to be replaced by the small buggy or cart. This in turn is being replaced by spouting systems, and it is the expectation of the writer before long to see concrete successfully installed by means of pumps.

One of the features which perhaps has received more study than any other part in the construction of a reinforced concrete building, has been the forms. Many steel forms have been made and used with more or less success, but until school buildings can be standardized and the forms made to reproduce moldings and architectural features, and where the forms can be used over and over again, this method will hardly prove successful. It is a problem, however, worthy of the best engineering talent and it is hoped will be successfully solved within the near future.

The unit cost of a reinforced concrete school building varies somewhat with the size and arrangement; and according to the relative amount of floor space actually occupied by the class rooms. This varies from twenty to forty per cent., all floors considered. The price per cu. ft. will vary from 17 to 20 cts. while the cost per sq. ft., all floors considered, will vary from $2.00 to $3.00, and per class room from $4,000 to $6,000, decreasing as the building increases in size.

It is difficult and unsafe to estimate buildings by unit prices. For instance, the Chatham School, a brick and wood building of the same number of class rooms as the Central Avenue School at Madison, shows the same price per cu. ft., with the price per sq. ft. less for the brick building than for the concrete building, but with price per class room the same for the both buildings. This was largely due, as before mentioned, to the economy in the floor space of the concrete over the brick building. In the case referred to, the brick and wood building actually costs the same as the one of reinforced concrete; from the price per cu. ft. there would seem to be no difference and from the price per sq. ft., the cost would be in favor of the brick building. While unit prices are interesting to the contractor as comparisons, they are very unsatisfactory as a basis for estimating.

USE OF REINFORCED CONCRETE IN SAN FRANCISCO AND VICINITY.

By John B. Leonard.*

In order to properly appreciate the use of reinforced concrete in San Francisco and vicinity at the present time, it is essential to understand the conditions existing in this territory immediately previous to the great earthquake of San Francisco in 1906. By vicinity, the writer refers to that portion of California that is within the radius of 150 miles of San Francisco.

Municipal Laws of San Francisco.

Prior to April 18, 1906, the building laws of the city of San Francisco were so framed that they did not permit the construction of an all reinforced concrete building. They did allow the use of reinforced concrete for foundation work, sidewalk walls and floor slabs. Architects were also allowed the use of reinforced concrete for intermediate beams in structural steel frames, and this device had been resorted to in two or three instances. There were many buildings in which reinforced concrete floor slabs were used, and also concrete fireproofing for the columns, but a large portion of the practice used tile for floors and fireproofing purposes. The advocates of reinforced concrete, however, were making a very strenuous fight to secure permission to construct of this material, and their efforts would undoubtedly have secured favorable results in a short time.

San Francisco being the metropolis was then, as it is to-day, the guide for the smaller interior cities and villages in all essential matters pertaining to building construction.

The Effect of Earthquake on Reinforced Concrete Buildings.

Owing to the building laws of San Francisco denying the privilege of constructing reinforced concrete buildings in the city

*Consulting Civil Engineer, San Francisco, Cal.

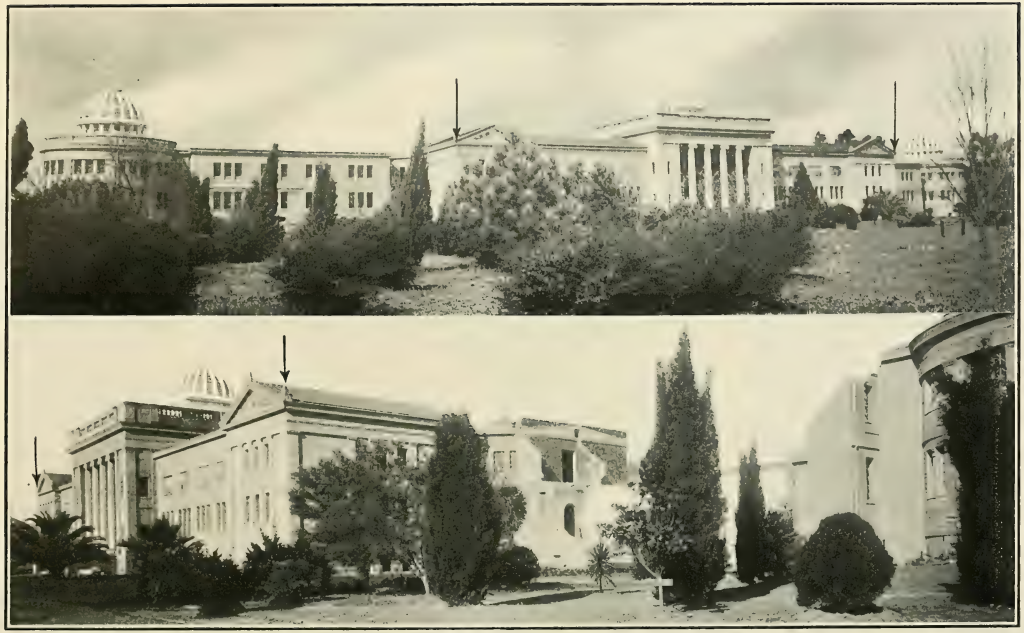

FIG. 1.—MUSEUM BUILDING, LELAND STANFORD, JR. UNIVERSITY, PALO ALTO, CAL., BEFORE AND AFTER EARTHQUAKE. THE UNINJURED REINFORCED MAIN BUILDING IS INDICATED BY ARROWS, AND THE COLLAPSED BRICK WING IS SHOWN IN LOWER VIEW.

limits, there was no completed building of this type in San Francisco at the time of the earthquake and fire. There was one building just being constructed. The owner had never received an official permit for the erection of this building, but being confident of the merits of reinforced concrete, intended to abide by the issue in the courts. This structure was intended for storage purposes and was to be six stories high. The footings, columns, girders, floor and roof slabs were to be of reinforced concrete, and the enclosure walls of brick. At the time of our great disaster, the second floor had been completed and forms were up for the third floor, also the curtain walls were up nearly to the third floor.

The building was not subjected to any intense fire, but it was in a section of the city that suffered severely from the earthquake. An examination of the building a few days after the disaster showed that the brick curtain walls had suffered considerably, but that the reinforced concrete portion of the structure was entirely sound. The faults in the curtain walls have since been corrected and the building carried on to completion in reinforced concrete.

At the Leland Stanford, Jr. University, located at Palo Alto, about thirty miles south of San Francisco in the Santa Clara valley, and but a few miles from the famous fault line, there existed a reinforced concrete building that was constructed some 23 years ago. This building, known as the Museum Building, Fig. 1, has a length of about 300 ft., and a depth of about 50 ft., and is two stories high. The construction was entirely of reinforced concrete, consisting of interior columns, girders, beams and floors, and enclosed with concrete bearing walls. The enclosure walls, however, were not reinforced except at the floor levels, the space between the floor levels consisting of plain concrete.

Shortly after the completion of the building, as was to be expected, shrinkage cracks appeared in some of the walls. Recent observers have mistaken these shrinkage cracks for earthquake cracks, but there is ample testimony of unquestionable character to prove that they were in existence years before the earthquake occurred.

It became the writer's privilege about a month after the shock to make a thorough minute examination of the structure in com-

pany with an engineer and an architect. Because of our realization of the important part that reinforced concrete would probably take in the reconstruction of the stricken section, our examination was made with unusual care. At the completion of the task, it

FIG. 2. — MEMORIAL ARCH AT LELAND STANFORD, JR. UNIVERSITY, BEFORE THE EARTHQUAKE.

was the unanimous opinion that this structure had suffered no structural damage from the earthquake. It was not subjected to any fire test. Columns, girders and floors showed no cracks or defects of any character, whatever. The only item of damage that could be found in the building at all, was a hole that had been

punctured in one of the floors, caused by the falling of a piece of statuary.

That the building had been subjected to a severe vibratory motion was evident in the main entrance. This entrance had a marble wainscoting about 4 ft. high, and the floor was covered with chips and dust caused by the grinding of the edges of the marble used in the wainscoting. Some of the chips were as large as a thumb nail.

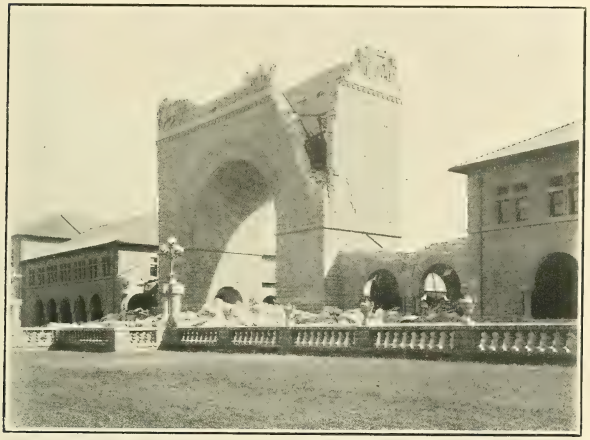

FIG 3.—MEMORIAL ARCH AFTER EARTHQUAKE, SHOWING EFFECT ON BRICK AND STONE STRUCTURES.

There was just being completed an addition to the Museum Building, this addition being almost ready for occupancy. The construction of the new work consisted of brick bearing walls and concrete floors, the concrete floors being supported by steel girders and columns. All the new work was badly wrecked by the earthquake. It is interesting to know that the University authorities had abandoned the reinforced concrete construction, and resorted to this type as being safer. The alarm was caused by the shrinkage cracks that existed in the main Museum Building.

This building certainly possesses for advocates of reinforced concrete more interest than any structure in this vicinity, on ac-

Fig. 4.— Sheldon Building, First and Market Streets, San Francisco, Cal. Constructed exclusively of reinforced concrete. B. M'Dougall, Architect.

count of the unquestionably severe ordeal through which it has passed. It will convince all fair-minded persons that a properly constructed reinforced concrete building of such dimensions, is a

safe and sound structure for earthquakes of as great magnitude as that which occurred in California in 1906. It was the intention and object of Governor Stanford when he authorized the construction of this building to have on the campus a structure, whose strength and durability would be the maximum that could be obtained. That he succeeded, can be doubted by no one who saw the campus after the earthquake.

The writer has heard it stated, and is informed that this state-

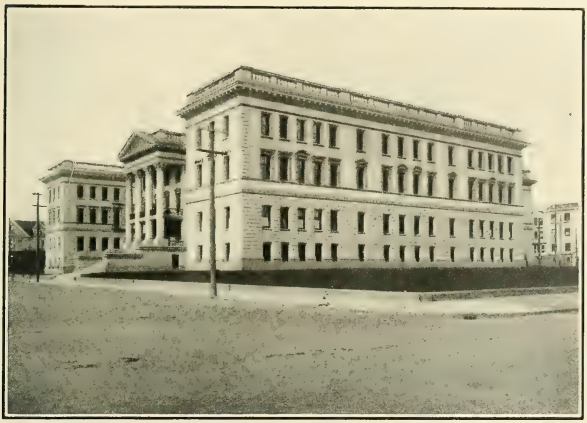

FIG. 5.—SOUTHERN PACIFIC HOSPITAL, SAN FRANCISCO, CAL.
D. J. PATTERSON, ARCHITECT.

ment exists in print, that the Memorial Arch, Figs. 2 and 3, at the Leland Stanford, Jr. University, was a reinforced concrete structure, and that it was a complete wreck. I take this opportunity to refute this statement, insofar as it pertains to reinforced concrete. The arch was a wreck, but it was constructed of brick.

Fire Resisting Qualities of Reinforced Concrete.

One of the great misfortunes for us is that there did not exist at the time of the fire, an all reinforced concrete building in San Francisco. Such information as was gleaned from the San

Francisco fire on this subject was derived wholly from the behavior of the concrete floors and fireproof columns in some of the steel frames. As in all other places, good, bad and indifferent concrete work had been done, so that the results must of necessity be variable in character. The same faults of workmanship existed, no doubt, in the buildings in which tile was the fireproofing element.

Fig. 6.—Pier 54, San Francisco Water Front, Constructed Exclusively of Reinforced Concrete.

Rebuilding in 1906.

When San Francisco faced the problem of rebuilding, it was a source of astonishment to those interested in construction to see the attention and investigation that was being given to reinforced concrete by the laymen. Public opinion became so strong for this material, that the Board of Supervisors were compelled to recognize its merits and permit its use for all building operations.

Its superior fire resisting qualities were recognized by comparing the general condition of such buildings as had used concrete

with those which had used tile. The result is that to-day in San Francisco concrete has been used as fireproofing for the steel frame buildings almost exclusively. There is but one case in the city in which tile was used. A typical reinforced concrete building is shown in Fig. 4.

Up to June 30, 1910, there had been issued in San Francisco permits for 132 reinforced concrete buildings. Several permits have been issued since, so that the number to date can safely be taken as over 140. Of this number, 128 are completed structures and being occupied, eight more will be completed within two months, and the balance within six months. The private capital invested in buildings in San Francisco from April, 1906, to June 30, 1910, as shown by the official reports of the California Development Board is as follows:

Class of Building	Number	Cost
Class A (steel frames)	105	$ 23,467,260.00
Class B (reinforced concrete)	132	9,059,371.00
Class C (mill construction)	1,739	53,597,631.00
Frame	15,844	63,701,643.00
Alterations	9,491	11,302,231.00
	27,311	$161,128,136.00
Addition of 15 per cent. for undervaluation		24,169,220.00
Total expended to January 1, 1910		$185,297,356.00
Municipal work inclusive of bond issue City work applies only to contracts actually awarded before January 1, 1910		6,150,371.00
		$191,447,727.00

The concrete buildings constructed have been built by a most conservative capital. Because of the known conservatism of the company, I mention in particular, the Southern Pacific Railroad Hospital, Fig. 5. This structure cost the railroad about $550,000 and was built of reinforced concrete with brick curtain walls.

The concrete buildings in San Francisco have been constructed in the face of one very great obstacle. For some reason, unknown to the writer, the banks have absolutely declined to negotiate loans for this type of construction. Many owners have built in structural steel, where they would have used reinforced concrete but for this difficulty.

State of California.

The State of California through the State Engineers Department, Nat C. Ellery, engineer, has used reinforced concrete very extensively in the replacing of such state buildings as were destroyed by the earthquake. The State Insane Asylum, situated at Agnews, a place about twenty miles from the Leland Stanford Jr. University, has been rebuilt of reinforced concrete. The old

FIG. 7.—REINFORCED CONCRETE RESIDENCE IN MENLO PARK, SAN MATEO COUNTY, CAL., FOR MRS. M. P. PAYNE. WILLIAM CURLETT AND SON, ARCHITECTS.

structures were entirely demolished in the earthquake, having been built of brick with timber interiors.

Very extensive additions to the State Penitentiary at San Quentin, situated on the bay but a few miles from San Francisco, were made of the same material.

The State Board of Harbor Commissioners.

The State Board of Harbor Commissioners are a body of men, who control the improvements of the harbor of San Francisco, and are appointed by the Governor for this specific purpose. Since

1906 they have used reinforced concrete very extensively in the construction of wharves. Piers 36-38 and 40 are founded on concrete cylinder piers, which support a series of steel girders. These girders are encased in concrete, and on them rests a reinforced concrete slab which supports the entire traffic of the wharf. Pier 54, Fig. 6, which is now under construction and nearly completed, is supported by concrete cylinders. The floor system of this pier, however, consists of a series of girders, beams and slabs, all of which are constructed entirely of reinforced concrete. This is the last

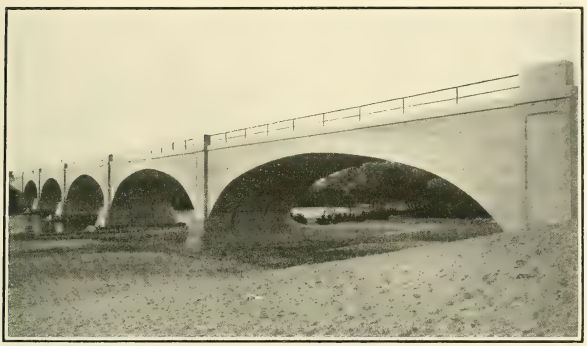

FIG. 8.—REINFORCED CONCRETE BRIDGE ACROSS SAN JOAQUIN RIVER, BETWEEN MADERA AND FRESNO COUNTIES, CAL. FIVE SPANS OF 80 FT. EACH.

pier constructed by the Board of Harbor Commissioners, but they now have in preparation plans for another one that will incorporate the same type of construction as Pier 54. These four piers have a floor area of about 360,000 sq. ft., and cost about $1,090,000.

It is beyond question that our State Board of Harbor Commissioners are being guided by their experience and investigations in the selection of reinforced concrete for permanent structures on the water front of San Francisco.

San Francisco.

The city of San Francisco has shown its confidence in reinforced concrete in a very emphatic manner. It is well known, no

doubt, that the great fire of San Francisco was made possible because of the breaking of the water supply pipes on the marsh just south of the city, thus leaving the fire department without water to combat the conflagration. Before a rebuilt city could be made safe, it was patent to all that some device must be used to protect the city in case the supply pipes should again be broken. It was finally decided to build storage cisterns distributed throughout the

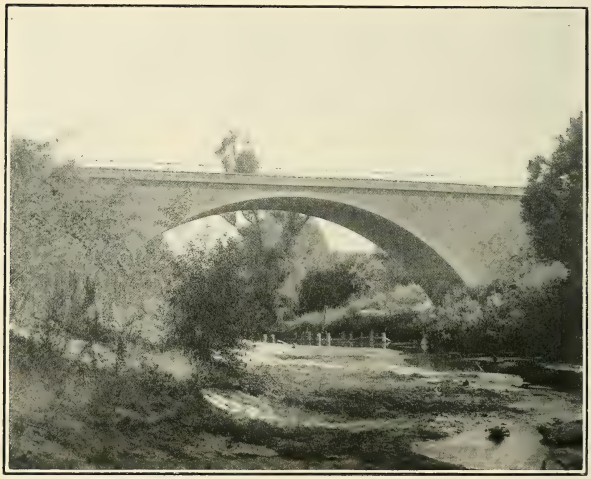

FIG. 9.—REINFORCED CONCRETE BRIDGE ACROSS DRY CREEK, STANISLAUS COUNTY, NEAR MODESTO, CAL. CLEAR SPAN 115 FT.

city at proper intervals. There have been 80 of these cisterns built in San Francisco, and all of them are constructed of reinforced concrete.

In the new sewer system for which bonds have been voted in the sum of $5,000,000, reinforced concrete is being used exclusively, where the sewers are of sufficient size to warrant. There has already been expended in these reinforced concrete sewers $1,500,000, and the balance of the bond money will be used for this same type.

The county infirmary building is of reinforced concrete, and in the steel frame buildings that have been constructed by the city, it is used exclusively for the fireproofing and floor constructions.

Vicinity.

The interior cities of Sacramento, Stockton, San José and Santa Rosa have each contributed several structures in reinforced concrete, and in each case the structures are of particular importance in these respective cities. Of the above mentioned cities, San José and Santa Rosa were severe sufferers from the earthquake and fire. In the city of Oakland are several very large concrete buildings, all built since April, 1906. The City Council of Oakland have just awarded the contract for the construction of an all reinforced concrete wharf.

A reinforced concrete residence is shown in Fig. 7.

Costs.

For the purpose of conveying to you an idea of the relative costs of a reinforced concrete building, and a structural steel frame building, San Francisco is selected as the location of the structure. The prices for labor and material existing at the present time are as follows:

Cement workers:	Laborers	$4.00	per day
	Finishers	6.00	per day
Housesmiths:	Inside workmen	4.00	per day
	Outside workmen	5.00	per day
Carpenters		5.00	per day
Cement		1.90	per bbl. delivered
Form lumber		15.00	per M. "
Gravel or sand		1.35	per yd. "
Crushed rock		1.50	per yd. "
Reinforcing steel		50.00	per ton

The building selected is one of six stories and basement, with a floor area of 10,400 sq. ft. on each floor, and a cubiture of 1,088,000 cu. ft. As actually constructed, the frame is of steel with reinforced concrete foundations, basement walls, floor slabs, and intermediate beams. The curtain walls are of brick. The total contract price for the brick work, concrete work, and steel frame

was about $104,000. This figure was considered very low. The cost of a reinforced concrete frame and curtain walls under the same conditions would not be more than $86,000. A concrete building would have made a saving of $18,000, or a little more than 17 per cent. It must be borne in mind, however, that the steel frame building had intermediate reinforced concrete beams, which, of course, reduces its costs to a minimum thus making the above comparison as favorable to steel as possible.

Bridges.

Previous to 1906 and within the territory included in this discussion, there was but one reinforced concrete bridge of any magnitude. This one was a structure consisting of ten 75 ft. spans, built by interior counties for highway purposes. This bridge caused considerable inclination among the county officials to weigh more carefully the merits of concrete bridges. By persistent efforts, the concrete men are being able to gradually convince the authorities that the extra cost of concrete bridges over steel is money well expended. The result is that there exists to-day over fifty reinforced concrete county bridges ranging in magnitude from 35 to 130 ft. spans in a section that four years ago contained but one. There is now being constructed by one of the interior counties, a bridge consisting of seven 195 ft. 6 in. spans. Typical reinforced concrete bridges are shown in Figs. 8 and 9.

The county surveyors, who under the laws of the State of California, control the designing of bridges for the counties, except through a special act of the Board of Supervisors, are rapidly becoming advocates of the use of reinforced concrete. The bridges that have been built in the interior which embrace flat slabs, girders and arches have done an immense work toward educating the people to understand the possibilities and utility of this material. Two counties of California have through their Board of Supervisors and by resolutions of the latter, committed themselves to the use of reinforced concrete for bridge purposes wherever possible.

THE TAYLOR CONCRETE COAL BREAKER.

By R. D. Richardson.*

A new field in the use of concrete is being developed by the construction of coal breakers and washeries in the Anthracite Region of Pennsylvania. It has been the usual practice to construct these breakers and washeries of timber, but there are certain serious disadvantages in timber construction to the extent that the large coal operators are beginning to experiment with other building material.

To those of you who are not familiar with breaker construction, a word with reference to their functions might not be amiss in order that a clearer idea may be obtained of the problems involved. The method of preparing coal for market varies according to the kind and grade of coal being mined, consequently no two breakers are exactly alike; but the general scheme is the same.

The coal in various sizes is conveyed from the mouth of the mine by the means of an inclined conveyor, either bucket or scraper line, to the head or top of the breaker. From the conveyor it runs by gravity to head shaker screens, the lump coal passing over the top screen goes to a picking table where picker boys take out all the rock and slate possible, then continues to the main rolls, where it is broken to grate coal size and under. From the main rolls the coal passes to two main shaker screens, grate coal passing over the top screen, egg coal over the bottom while stove and under passes through the bottom screen going to three counter main shaking screens, which separate stove, chestnut and pea from the sizes smaller. Sizes under pea go to a conveyor and elevator to the Washery for further preparation. The grate and egg coal passing over the main shaking screens mentioned above goes direct to mechanical pickers, where more slate and impurities are separated from the coal, then to the pockets. Similarly the stove and nut from the counter main shaking screens goes to mechanical

* Concrete Engineer, Coal Mining Department, Delaware, Lackawanna and Western Railroad, Scranton, Pa.

pickers, and then to the pockets. The pea goes direct to pockets without being picked.

Returning to where the coal first entered the breaker, namely, the head shaker screens, the grate and egg goes to mechanical pickers, thence to the pockets; or if occasion demands, goes through pony rolls and more shaking screens for sizing before reaching pocket. In the Washery the coal is sized and purified by means of shakers and jigs. The smaller sizes will not run by gravity except at a very steep pitch, hence considerable water is used before coal reaches the pocket. More water is used in jigging.

The great number of shakers, screens and mechanical pickers together with the rolls moving rapidly cause much vibration. The shakers are operated by the use of cams and while not representing great weight, still moving back and forth 160 times per minute are instrumental in causing most of the vibration. The main rolls weigh 36,000 lbs., and have a velocity of 96 revolutions per minute.

The ordinary breaker is from 100 to 150 ft. high, with shakers and rolls in the upper part; and it is not surprising that the timber breakers with their loose joints should show an appreciable vibration. The inclined conveyor is usually entirely separate from the breaker, and after standing in the conveyor head frame it seems surprising that the timber breakers stand up as long as they do. The water has, of course, a deteriorating effect on the timber, which usually shows in the pockets first. The ordinary life of a timber breaker, before it is necessary to make extensive repairs, is from six to ten years. When repairs are started it may appear that only a few timbers need to be removed; but often leads to as extensive overhauling as building a house around a keyhole. In many cases it would have been economical to build it entirely new. Thus, it is not surprising that attention is being given to other building material than timber.

In 1905 the Pine Hill Coal Company, at Minersville, Pa., built the pockets of concrete and the upper part of timber. It was a very massive design, consisting of a series of arches from bent to bent, and probably expensive, but, so far as the writer is able to learn, satisfactory. In 1908 the Hillside Coal and Iron Company built a concrete Washery to a height of 55 ft. above loading tracks, this being 17 ft. above top of pockets. During the last year the Delaware, Lackawanna and Western Railroad

Coal Mining Department constructed a breaker at Taylor, Pa., Fig. 1, which is entirely of concrete, and have under construction a concrete Washery at the present time. The Lehigh Valley Railroad has two and the Hillside Coal and Iron Company awarded a contract in November, 1910, for another. These last mentioned, with exception of Taylor, are of concrete to top of pockets or one bent above; steel frame work has been used in a few cases.

The Taylor Breaker is 107 ft. 6 ins. wide and 133 ft. long, and 167 ft. from bottom of foundations to highest point of roof.

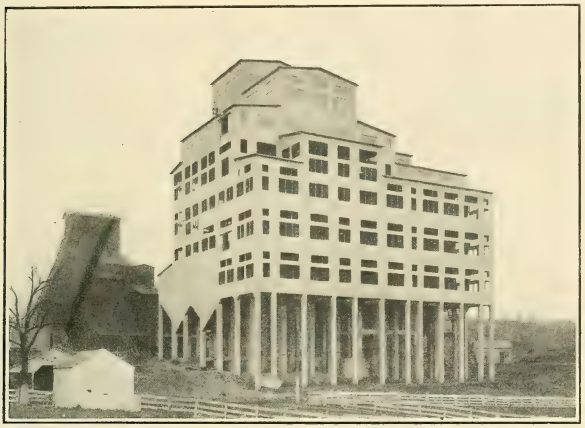

FIG. 1.—REINFORCED CONCRETE COAL BREAKER, COAL MINING DEPARTMENT, DELAWARE, LACKAWANNA AND WESTERN RAILROAD, TAYLOR, PA.

The building is divided into 6 bays, Fig. 2, for future reference the rows of columns will be designated as A, B, C, D, E, F and G, with column spacing of 20 ft., 13 ft. 6 ins., 20 ft. and three bays of 18 ft. each. There is a row of pockets 53 ft. 6 ins. wide extending from A to D, running the whole length of the building. Transversely, Fig. 3, there are 12 bents, most of which are 13 and 15 ft. on centers. There are intermediate columns in rows A, B, C and D, under the pockets, making the columns 6 ft. 6 ins. and 7 ft. 6 ins. centers. The pockets have partitions 6 ft. 6 ins. and 7 ft. 6 ins. centers, 17 pockets in all to accommodate the different

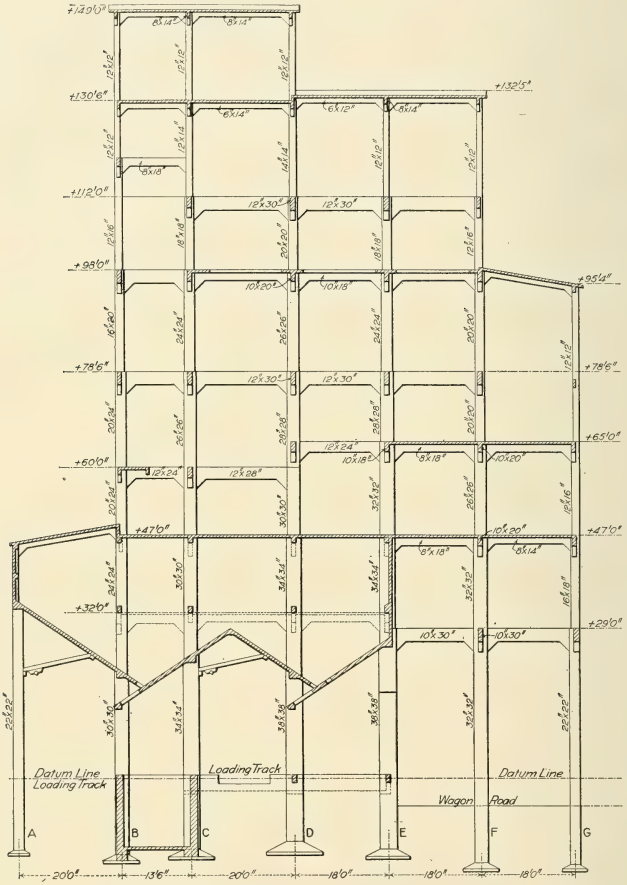

FIG. 2.—SECTION OF REINFORCED CONCRETE COAL BREAKER, TAYLOR, PA.

sizes of coal; 2 for grate, 2 for egg, 3 for stove, 3 for chestnut, 4 for pea and 1 each for barley, rice and buckwheat. The pockets are about 30 ft. in maximum depth and have a combined capacity

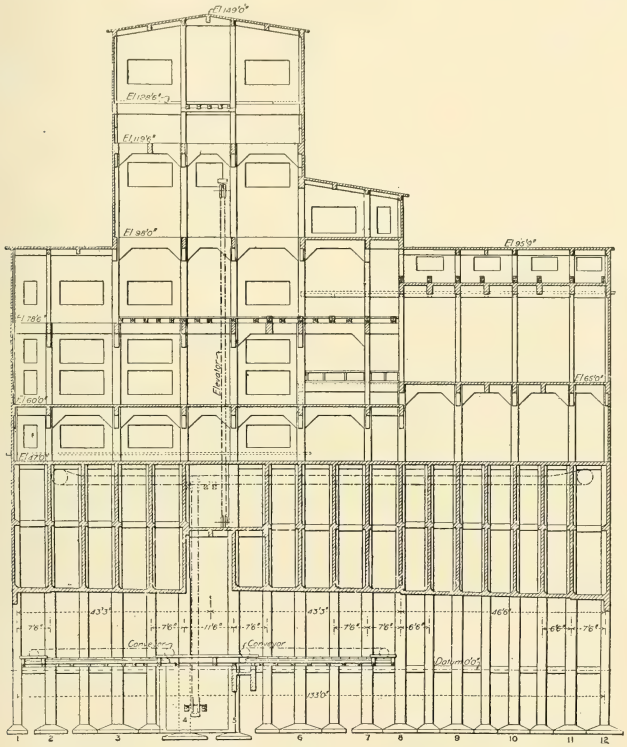

FIG. 3.—TRANSVERSE SECTION OF REINFORCED CONCRETE COAL BREAKER, TAYLOR, PA.

of about 3600 tons. Two loading tracks run underneath, between rows A and B, and C and D on a 1.5 per cent. grade, the bottom of pockets being built at same grade for greater convenience in

loading either box or hopper cars. Fig. 4 shows the breaker during construction.

The foundations extend below base of rail about 18 ft., the depth necessary to secure good foundations on clay. Borings indicated 16 ft. of culm and ashes, 3 ft. clay, 17 ft. gravel, clay and sand, 2 ft. rock and then a vein of coal. Tests were made on the clay foundations by loading 5 sq. ft. with pig iron and reading deflections with level. The soil was first loaded with 1.5

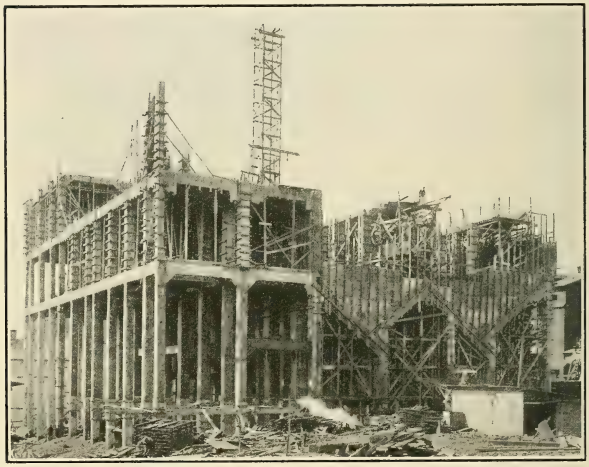

FIG. 4.—TAYLOR COAL BREAKER UNDER CONSTRUCTION.

tons per sq. ft. and showed an immediate settlement of 3/32 ins., and in 18 hours, 1/8 ins. A load of 5 tons per sq. ft. showed an immediate settlement of 11/32 ins., and at the end of 24 hours 7/16 ins. Judging from readings it was considered safe to design footings using bearing value of 3.5 tons per sq. ft. Under the pockets, footings were designed as continuous, running the entire length of breaker, except where elevator pits required greater depth, in which case mattress foundations were used. The two rows F and G were designed as isolated footings.

In specifying loads for upper part of structure it was, of

course, necessary to take into consideration the weight of moving machinery and resulting vibration. This is largely a matter of judgment, but the chief aim was to obtain a structure sufficiently massive to absorb all vibration, and for this reason a live load in most cases of 200 lbs. per sq. ft. was used on floor slab with 100 per cent. impact figured in all beams, girders and columns, except at such places where loads were known. Where known loads existed they were specified, using the 100 per cent. impact.

All the machinery rests on heavy timbers which are anchored to beams by bolts, and thus it was not considered necessary to have floor slabs figured for a load heavier than 200 lbs., a load which would be obtained by piling coal 4 ft. high, a case which often happens in breakers. Anchorage for the timbers was provided for by inserting gas pipe in the beams and bolting clear through unless this arrangement was hindered by knee brackets. In this case anchor bolts were placed in the concrete. Roofs were designed for a live load of 40 lbs. per sq. ft. Floor elevation 128.5 above track for 150 lbs. per sq. ft. No machinery rests on this floor. Floor elevation 112.0 for 200 lbs. per foot with 100 per cent. impact in all beams, girders and columns. Floor elevation 98.0 the same and also to be able to support main rolls, weight 36,000 lbs. in any panel on any part of the floor. Floor to be designed for that position of load which produces maximum stresses and in addition one panel, BC45, to carry extra load due to elevator, amounting to 16,200 lbs. Impact 100 per cent. in all beams and columns. Elevation 78.5 has no floor slab, but isolated beams designed for shaker screens, weighing 32,000 and 38,000 lbs., assuming ¾ weight of screen on head and ¼ weight on tail end. Impact 100 per cent. Largest girder section obtained on this floor to be used uniformly throughout the elevation. Floor elevation 60.0 and 65.0 were designed for 200 lbs. live load, except panels containing jigs weighing 50,000 lbs. resting on four stringers equally loaded, spaced 4 ft. apart. These loads were placed on girders in that position giving maximum stresses. Impact 100 per cent., as in other floors. Floor elevations 47.0 and 29.0 designed for similar loads.

In designing pockets it was assumed that they would be filled with coal to elevation 47.0. Column loads were reduced at each successive floor. At elevation 128.5 live loads and vibration were

reduced to 95 per cent. At elevation 112.0 all live loads and vibration above were reduced to 90 per cent., and so on down to elevation 47.0, where all live loads and vibration were reduced to 75 per cent.

Low working stresses were used, the concrete being figured at 500 lbs. per sq. in., due to flexure and the reinforcement not to exceed 15,000 lbs. per sq. in. in tension. In figuring shear 50 lbs. per sq. in. was considered to be taken by the concrete, and the remaining amount carried by stirrups figured at 10,000 lbs. per sq. in. Concrete in columns was designed for 400 lbs. per sq. in., reinforced with longitudinal bars figured at 10,000 lbs. per sq. in.

The pockets are the most intricate part of the design and offer the greatest opportunity for ingenuity by the designer. Each partition is designed for full coal pressure on each side, as it is quite possible that one pocket might be full while adjacent ones are empty. It was assumed that the angle of friction between coal and concrete was 27 deg. Each partition was designed to carry the floor over the pockets, its own weight, together with the weight of coal transmitted due to friction without using any beams underneath, thus exposing a perfectly flat surface except at the two ends, where beams were used. This feature of design was accomplished by placing a 1 in. bar at the bottom of each partition, extending from column to column, resting in a hooked bar 1 in. sq., 2 ft. 6 ins. centers extending vertically in partition walls about 10 ft. By this method shear is the main force to be reckoned with. In taking care of the lateral pressure of the coal lateral beams dividing partition wall into two spans were placed, running horizontally from column to column. These beams were bevelled on upper side to prevent lodgement of coal and were 22 ins. wide, reinforced both sides with three $1\frac{1}{4}$ in. bars wrapped with collars. This allowed the designing of partition walls as plates supported on four sides. Walls are 7 ins. thick, reinforced with $\frac{1}{2}$ in. square bars vertically and horizontally on both sides except end walls below lateral beams and 6 ins. above. Tie rods run in the opposite direction to the lateral beams to take the reaction and are hooked around the bars in columns which were designed for bending.

The slab at the bottom was designed for that depth of coal

which gave maximum moment. The partitions being very deep in comparison to their width, the major portion of the load is transmitted direct to partitions due to friction and arching. The weight of coal resting on bottom would, of course, be the difference between the total weight of coal in partition less the amount going to partition and was not maximum when the pockets were completely filled. The slab, 7½ ins. thick, is reinforced with ½ in. square rods 8 ins. on centers bent up over the 1-in. rod running in bottom of each partition wall.

By this method of pocket design it was necessary to pour partitions along with the slab at bottom of pocket in order to make slab self-supporting. As a matter of fact, the partitions were poured up to the lateral beam at the first operation, and forms were removed under bottom as soon as this had set properly, regardless of whether remaining portion of the partition had been poured, as this provided more than sufficient depth of beam to carry dead load. Small concrete blocks 7½ ins. long were made in advance, having ends on a bevel of 8 ins. on 12 ins., the pitch of the bottom of the pockets, and leaving the point of a nail projecting from one end. These were used in supporting upper forms of the bottom slab as well as partition forms. The nail sticking in the bottom forms kept block from sliding on the 8-in. pitch. The partition forms were placed before the top forms of slab and placed in alternate pockets, first allowing men to place the reinforcement in partition, and fasten to forms in vertical position. As soon as the reinforcement was placed in two partitions and in bottom slab, carpenters placed remaining partition forms, and as concrete was poured top forms of slab were placed in 3 ft. sections.

On the whole, the field construction of pockets was laborious and difficult to perform. While the design contained a minimum amount of concrete for maximum efficiency, yet on account of the thinness of partitions and taking into consideration the fact that the combined thickness of all partition reinforcement totaled in some panels 3⅜ ins. in a 7-in. wall, it is quite probable that better results would have been obtained if a thicker partition were used and designed as a simple slab rather than as a plate, which would have required reinforcing in one direction only. It can be easily imagined that great care had to be taken in depositing

concrete where the steel took up 3⅜ ins. in a 7-in. wall to get a perfect job, free from voids.

Aside from the pockets the remaining portion of the structure offers no unusual features of design. Isolated beams 30 ins. deep were used at elevation 65 and 78.5 in the greater portion of the building, as it was impossible to locate all floor openings for numerous coal chutes, etc. At all other floor elevations the openings were indicated and a 5-in. slab was used. Knee brackets 2 x 2 ft. reinforced with two ½-in. square bars were used on practically all beams and girders framing into columns, also a great number of compression bars were placed in top of beams extending over supports to insure the structure being as rigid as possible. Roof slabs were all 3½ ins. thick with 6 ft. 6 ins. and 7 ft. 6 ins. spans reinforced with ⅜-in. bars 9 and 12 ins. on centers, and transversely with ¼-in. bars 12 ins. on centers for temperature stresses.

Curtain walls were all 5 ins. thick, set in 2 ins. from face of columns above elevation 47.0, a 2-in. recess being left in columns as well as ties every 12 ins. Walls rested 1 in. on beams at each floor level and ties were provided in sides of beams tying in sidewall to prevent any outward buckling. The beams were not designed to carry any weight of curtain walls but the weight of wall was considered transmitted direct to columns.

The columns at bottom vary in size from 22 x 22 ins. to 38 x 38 ins. reinforced with longitudinal bars and 14 gauge steel by 1-in. collars spaced 18 ins. as maximum and in no case farther than out to out distance of column bars. In bents A and G columns are 22 x 22 ins., and have no brace for a height of about 42 ft. or 22 diameters. Part of the columns in Bent A were poured in January, during fairly cold weather, and when forms were removed about 6 weeks later when weather was mild, rather alarming cracks appeared vertically in the center on all four sides, extending practically the entire length of column up to pockets. The exact cause of these cracks is a matter of conjecture, but fearing that it might be due to premature removal of forms, and on account of the great height of columns the clamps were replaced and left till late in Spring. These columns set up in time and apparently are as hard and safe as any of the others.

The mixing plant consisted of a ¾ cu. yd. mixer run by a

motor most of the time, dumping into automatic dump bucket discharging into pocket from which it was conveyed to forms by carts and wheelbarrows. The material was conveyed to the mixer by a single side dump tilting car, running on a narrow gauge track, pulled up incline by hoisting engine and then returning by gravity underneath a pocket divided into sand and stone compartments. The material pocket was placed about 6 ft. below railroad track, alongside of breaker and material was stored on each side of pocket. This plant was capable of handling a cubic yard of concrete every five minutes.

About 5,000 cu. yds. of concrete were used and over 500 tons of reinforcing bars. Tests showed the bars to have an elastic limit varying between 45,000 and 55,000 lbs. per sq. in., ultimate strength between 65,000 and 85,000 lbs. per sq. in.; elongation in 8 ins. of 15 to 25 per cent. Hooped steel, 14 gauge 1 in. wide and 18 gauge ¾ in. wide, was used for stirrup, looping under each bar in bottom of beam and also around compression bars in top. A standard Portland cement was used practically throughout, Tests show tensile strength of about 375 lbs., initial set in 3 hrs. and final set in 5 hrs., and the crushed stone was of very good quality, analysis showing 91 per cent. pure silica and testing high in compression.

The Breaker was designed and constructed by the Hennebique Construction Company, New York, N. Y., having in the field Mr. W. R. Twohig as Superintendent, until on account of sickness he was succeeded by Mr. H. A. Nugent, who completed the work.

Coal breakers and washeries designed of concrete are undoubtedly far superior to timber or steel construction in point of permanence and durability. Fire risk, which is very high in timber construction, is practically eliminated. However, there is one feature in concrete construction that is not entirely free from objection, namely, the abrasion of the coal on the bottom slab of the pocket, and the deterioration of the concrete due to sulphuric acid which exists in the mine water. This acid enters into composition with the free lime existing in the concrete and forms calcium sulphate. Calcium sulphate is insoluble, and as soon as a sufficient coating is obtained on the surface deterioration ceases; but as this compound is not as hard as concrete the sliding of the coal will probably keep it worn down, thereby leaving the con-

crete open to the action of the acid, and in time materially weakening the pockets. It is a much mooted question whether sulphuric acid of the strength found in mine water will affect the strength of concrete in cases where abrasion does not exist. However, the Delaware, Lackawanna and Western Railroad has a reinforced concrete dam, designed and constructed by the writer in 1907, which up to the present time shows no deterioration and holds water pumped from the mines. This is a problem that will undoubtedly be solved only by experimenting, and the engineers of the coal regions would appreciate any information that other engineers could give as a result of their experience on this subject.

THE USE OF REINFORCED CONCRETE IN SEA WATER.

By Raymond Baffrey.*

In view of the thorough manner with which the subject of concrete in sea water was covered at the last Convention,† it would seem appropriate to preface this paper with an apology for the failure to present any striking novelty or final solution of the problem. The somewhat conflicting theories entertained by persons qualified to speak with authority on the chemical aspect of the case renders it necessary for the ordinary engineer to approach the matter with considerable diffidence. It may be of interest, however, to review the consensus of opinion regarding the chemical and physical effect of sea water on concrete and to describe a few of the more important works constructed in European waters within the last 15 years, showing the wide range of application of reinforced concrete to maritime structures.

Several years ago, when the attention of the engineering profession was first called to the rapid development of the concrete industry, it was feared that reinforcement imbedded in concrete submerged in sea water would be subject to deterioration. To-day, however, it is generally admitted that reinforcement in dense concrete will not rust.

In 1907, a block of concrete was broken off from a pier constructed in the Bay of Audierne, France. The anchoring steel rod was in a perfect state of preservation, although it had been imbedded in the sea water since 1848.

Iron bolts in perfect condition were found in a piece of concrete chipped off a dry dock built in Brest, France, about 100 years ago.

In 1881, Mr. W. T. Douglas, Engineer of the Lighthouse Commission, England, when wrecking the Eddystone Lighthouse built in 1857, found a bundle of iron rods in perfect condition embedded in Aberthaw lime cement.

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*President, Hennebique Construction Company, New York, N. Y.

†See *Proceedings*, Vol. VI, 1910, pp. 120-179.—Ed.

In 1906 a part of a Hennebique pile, driven in 1898, was taken to the laboratory for examination. The steel was found in an absolutely good condition.

In the matter of concrete in sea water, we find the same theory opposed to the experience of the last twenty years. One firm alone has designed over two hundred docks and piers since 1898, aggregating some three million dollars worth of work and has not yet organized a department of chemical engineering.

It is impossible at the present time to state definitely what reaction takes place when Portland cement comes into contact with certain salts contained in sea water. We know that the mag-

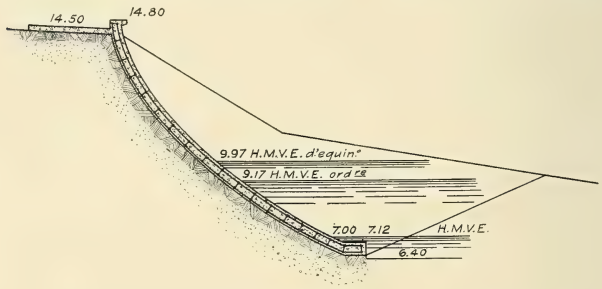

FIG. 1.—REINFORCED CONCRETE SHORE PROTECTION AT DIEPPE, FRANCE, CONSTRUCTED 1901.

nesium sulphate and the sodium sulphate in the sea water act on the free lime in the concrete to form calcic sulphate and calcic sulpho aluminate. In a porous concrete the formation of the salts, with the corresponding increase in volume, causes a certain amount of expansion and resultant checking of the concrete surface similar to the effect of frost on porous stone. The mechanical action of the waves accelerates the abrasion of the surface and therefore concrete structures in seas subject to tide suffer more than those in still water.

Two ways have been suggested to avoid the chemical action of the sulphates contained in sea water on the free lime and the aluminates contained in the set cement. *First,* it has been proposed to add to the cement some chemical compounds, which in

combining with the free lime would form a compound free from the action of the sulphates; this has led to the addition of puzzolona, trass, etc. *Second,* it has been proposed to eliminate the aluminum from ordinary Portland cement and replace it by iron which would avoid the formation of the calcic sulpho aluminate, supposed to be the ill effects in concrete. However, the situation is further complicated for us by the fact that the sea water varies in different parts of the continent and also by the presence of sewerage near the large cities on the sea coast. By using a compact, impervious concrete, well set, the action can be eliminated.

It is obvious that in order for a chemical reaction to take place the molecules of the mixture must be in intimate contact,

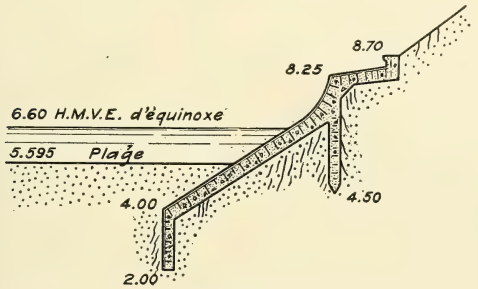

FIG. 2.—REINFORCED CONCRETE BEACH SHORE PROTECTION AT BELLE HENRIETTE, FRANCE.

hence if concrete is impervious to water, the action will be confined to the outside surface. In fact, as is seen in such concrete stuctures subject to sea water for over 15 years, the action is absolutely nil.

Suggestions as to incorporating waterproofing compounds in the concrete or painting the outside surface with coal tar preparations have not given as uniformly satisfactory results as a rich and well graded concrete of medium consistency.

Too much stress cannot be laid on the absolute necessity of grading the sand and stone to make a dense concrete, varying the proportions, if necessary, as the product of different crushers or gravel banks is incorporated in the work. It should also be noted

that nearly all important work in England and France in recent years has been constructed with a very rich mixture, generally about 1 : 1½ : 3.

It is universally admitted that a medium or quaking consistency of the concrete is to be preferred to either the very wet or very dry mixtures and furthermore it is essential that reinforced

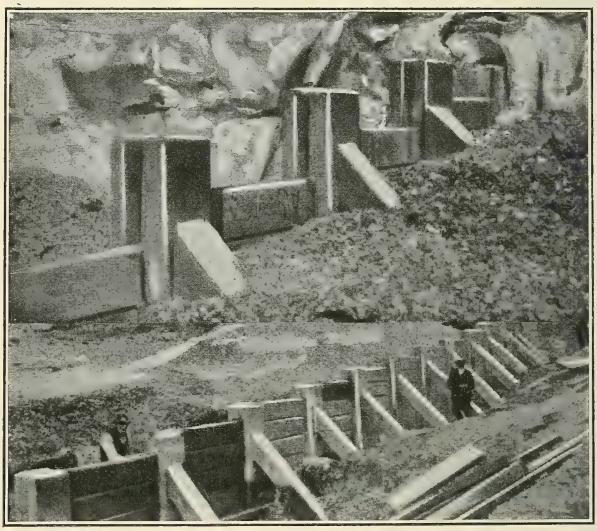

FIG. 3.—REINFORCED CONCRETE GROYNES FOR SHORE PROTECTION AT BRIGHTON, ENGLAND.

Upper view shows end of a Groyne at a cliff, illustrating also the accretion of shingle; lower view shows completed Groynes.

concrete in small sections should be well cured in air before being subjected to the mechanical effect of the sea water.

Sea water work can be classified under three different headings:

1. Work where mass concrete is used, such as gravity walls, locks, shore protection, etc.

2. Work where reinforced concrete piles and concrete sheet piling are used such as wharves, piers and jetties.

3. Reinforced concrete caisson work.

I. Mass Concrete.

Mass concrete has been used for centuries in sea water as ordinary masonry and its advantages from the standpoint of speed and economy have long been recognized. In Italy to-day we find sea walls built by the Romans with a concrete having as a

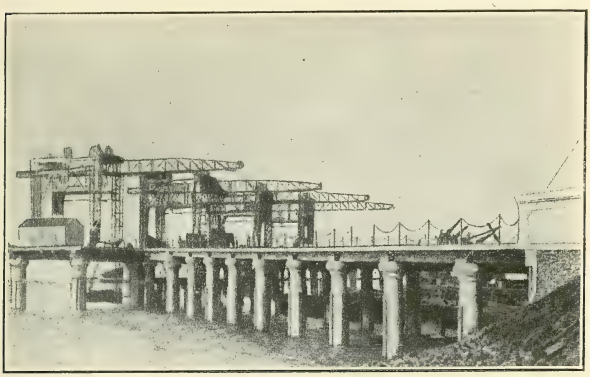

FIG. 4.—DAGGENHAM COALING JETTY, ESSEX, ENGLAND, BUILT OF REINFORCED CONCRETE.

binder puzzolona and lime cement. Concrete blocks have been cast on shore, transported and sunk in place in the same manner as granite.

The disadvantages of gravity walls are at once apparent, since the life of the superstructure is absolutely dependent on the foundation and the latter is subject to scour. It has been found more economical to design monolithic structures.

Shore protection works are generally much lighter, not being designed for earth pressure but rather for the protection of the coast from erosion.

A wall was constructed at Dieppe, France, in 1901, with a parabolic cross-section, Fig. 1, having a short vertical tangent at

the top to throw the water back into the sea. The height of the embankment is about 26 ft., the concrete being 16 ins. thick for the first 4 ft. of the height and 12 ins. thick for the balance. It is reinforced vertically and horizontally by a network of rods. Some time after the wall was built, a heavy storm washed away

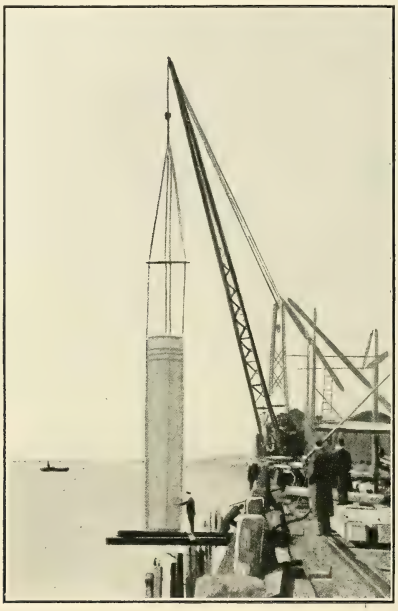

FIG. 5.—LOWERING A REINFORCED CONCRETE MOUCHEL CYLINDER IN DUNDEE HARBOR, ENGLAND.

the masonry wall built at the end of the reinforced concrete wall, while the latter withstood its action perfectly.

At Belle Henriette, France, a wall was constructed with a line of concrete sheet piles 12 ft. long, Fig. 2. The shore was covered with a slab of reinforced concrete supported at intervals by concrete piles.

A simple shore protection is shown in Fig. 3, grooved reinforced concrete piles support reinforced concrete shingles. These "Groynes" are erected near Brighton (England).

II. Reinforced Concrete Structures.

Reinforced concrete piles caused to make the greatest step in the use of reinforced concrete in sea water as wherever in a structure a wooden pile was used a reinforced concrete one could be substituted with the great advantage of obtaining permanency, simplicity and uniformity in construction.

The first concrete sheet piles were used at the Sables d'Olonne

FIG. 6.—REINFORCED CONCRETE KINGS DOCK AT SWANSEA, ENGLAND, BUILT FOR THE GREAT WESTERN RAILWAY.

in 1898, to reinforce an old masonry sea wall. The heads of the piles were capped with reinforced concrete beams anchored back to the masonry. The piles were 16×10 ins. in section, 19 ft. long and reinforced with six $\frac{7}{8}$ in. rods, the concrete being proportioned $1 : 1\frac{1}{2} : 3$. This work is in as good a condition to-day as it was at the time of construction.

In the same year a jetty was constructed at Southampton, the Woolston Jetty,* 136 ft. long, with a river frontage of 100 ft. The foundation consists of reinforced concrete piles braced at M. H. W. L. The deck was designed as an ordinary beam and girder floor. Ten years after its erection, the owners, Mordey, Carney and Company, stated that it was in as good con-

*An illustration of the Woolston Jetty appears on p. 133, Vol. VI, of the *Proceedings*.—Ed.

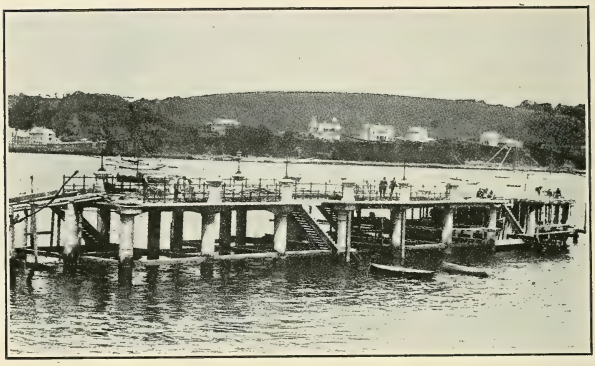

FIG. 7.—REINFORCED CONCRETE PRINCE OF WALES PIER, AT FALMOUTH, ENGLAND, DURING CONSTRUCTION.

FIG. 8.—REINFORCED CONCRETE WHARF AT NEW CASTLE-ON-TYNE, ENGLAND, SUPPORTING TURRET SHOP OF W. G. ARMSTRONG, WHITWORTH AND COMPANY.

dition as when first erected. "It had cost nothing for maintenance, and the only money spent on it was for the slight repairs entailed by a collision in 1901, when a large steamer crashed into it, but without causing anything more than local injury."

The fact that reinforced concrete piles can be made on shore with selected materials and subject to proper inspection has led to their adoption for foundations in sea water work in all parts of the world.

Fig. 4 shows the Daggenham Coaling Jetty, Essex, England, built in 1901. The jetty is 780 ft. long and 35 ft. wide with 78 ft.

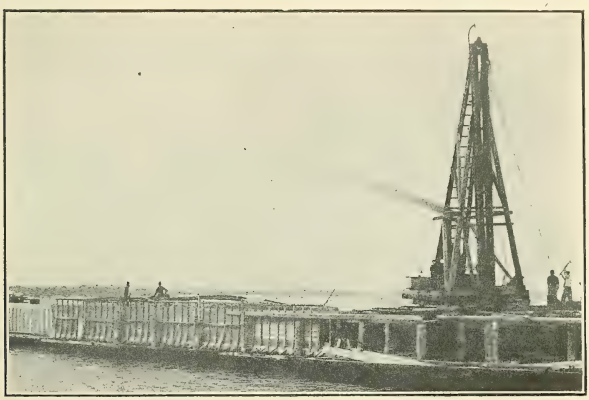

FIG. 9.—WOODEN PILES REPLACED WITH REINFORCED CONCRETE IN WHARF AT RAVENNA, ITALY.

between the river bed and the deck. The piles were driven in groups of three encased in reinforced concrete cylinders also filled with concrete. It carries railway rolling stock and four transporter cranes weighing 60 tons each.

The plant with which the cylinders were placed at the Eastern Wharf, Dundee, is shown in Fig. 5. The original wharf was 410 ft. long. Since then two extensions, one 380 ft. and one 310 ft., have been built. It might be noted in this connection that there is a 12 ft. tide at Dundee, and the climatic conditions are more severe than at Boston, Mass.

Fig. 6 shows the Kings Dock at Swansea, England, built for the Great Western Railway, 750 ft. long and 50 ft. wide. Another pier 1,100 ft. long has since been erected in the south side of the pier. This wharf has accommodations for three fixed and two movable tips.

The Prince of Wales Pier built in 1903, at Falmouth, England, is shown in Fig. 7. The construction is similar to that of the Daggenham jetty.

Fig. 8 shows a wharf built for W. G. Armstrong, Whitworth

FIG. 10.—REINFORCED CONCRETE AMUSEMENT PIER AT ARCACHON, FRANCE.

and Company, at Newcastle-on-Tyne, England. This wharf is 920 ft. long and 45 ft. wide, carrying a turret shop with a floor load of 900 lbs. per sq. ft.

A wharf was built at Ravenna, Italy, in which the wooden piles in the old structure were replaced with reinforced concrete piles, Fig. 9. A similar wharf was built in Nantes, France, in 1902, with piles 45 ft. long.

A good example of the light and graceful framing possible in reinforced concrete design is shown in Fig. 10, an amusement pier built at Arcachon, France, in 1902.

Fig. 11 shows the use of concrete sheet piles in the construction of sea walls at River Itchen, Southampton, in 1897, for the London and Southwestern Railway Company. The adjacent faces

of the sheet piles are provided with grooves. These are filled in, after driving, with cement grout, thus insuring a good joint between the piles.

Fig. 12 shows a reinforced concrete wharf at Pozzuoli, Italy, and Fig. 13 a lighthouse at Singapore, supported on concrete piles.

Concrete piles were used as reinforcement of the old quay walls at Boulogne. The masonry wall, constructed many years ago, showed a tendency to tilt under the pressure of the earth.

FIG. 11.—CONCRETE SHEET PILES USED FOR SEA WALLS AT SOUTHAMPTON, ENGLAND.

Its stability was retained by removing the earth fill and constructing a concrete floor supported on concrete piles, Fig. 13.

III. Reinforced Concrete Caissons.

Caissons of steel and wood are considered more as methods of construction than permanent works, adding nothing to the strength of the structure. On the contrary, concrete caissons can be designed to serve as permanent construction. They have taken the place of the immersed concrete blocks which were used some twenty years ago to form foundations on which breakwaters, quay walls, etc., were constructed. In open seas, these concrete blocks had to be of very large dimensions in order to resist the pounding

of the waves; hence it was advisable, in order to facilitate the placing, to make the blocks hollow, float, sink and fill them with concrete or other suitable material.

The breakwater of Madras has been constructed by means of cylindrical steel caissons 42 ft. in diameter and 53 ft. high, which were filled with concrete, in 2 operations. In the first one concrete was deposited so as to partially sink the caisson, 7 cylindrical openings being left through the mass; in the second one, these cylinders were filled.

At Heyst-Zeebrugge, Holland, the rectangular caissons were

FIG. 12.—REINFORCED CONCRETE WHARF AT POZZUOLI, ITALY.

82 ft. × 24 ft. 8 ins., and 29 ft. 6 ins. high; they were of steel divided into three cells by two transverse partitions. The caissons were sunk in place and filled with concrete.

At Rotterdam, Holland, a sea wall was constructed by using reinforced concrete caissons constructed in a dry dock, then floated and sunk in place, using concrete of lean mixture for ballast in order to give the necessary stability. These caissons were 35 ft. long.

At Touapse, Russia, a jetty was built with reinforced concrete caissons 56 ft. long and 21 ft. wide and 6 to 22 ft. high, averaging 250 tons when empty and 1,600 tons when filled.

In this country several reinforced concrete caissons were built during the last 2 years. One application consisted in constructing the piers of a yacht wharf at Glen Cove, L. I. The caissons were 12 × 12 ft. and 17 ft. high. The constructor used a compound which is a resinate of calcium and silicate of alumina; it generates a silicate of lime and silicate of alumina in crystalline

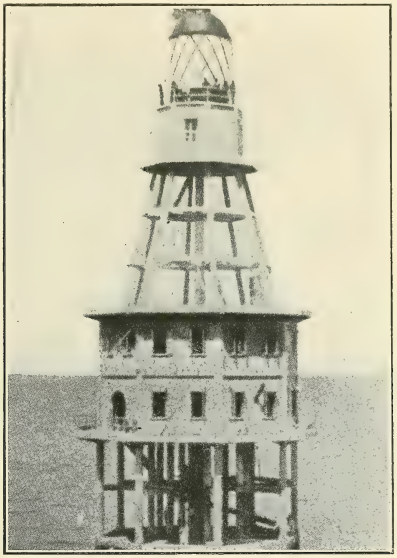

FIG. 13.—REINFORCED CONCRETE LIGHTHOUSE AT SINGAPORE, SUPPORTED ON CONCRETE PILES.

form, and both of these materials are immune from the action of sea salts. A lighthouse foundation was constructed at Huntington Harbor this summer by the same method.

By far the most striking example of reinforced concrete caisson work is the artificial island of the Battery des Maures, used as a torpedo testing station by the Schneider Iron Works in France. This is simply a large concrete box 77 × 55 × 51 ft. (See

Fig. 14.) In this case it was necessary to make an artificial island in a bay in order to obtain a stretch of still water several miles long—about 700 ft. wide and 35 to 40 ft. deep. The owners first planned to construct a large steel caisson and sink the same by means of compressed air, but no contractor would submit a proposal within reason in view of the possibility of the plant being destroyed by storm. It was next planned to build up an island by means of heavy artificial blocks but it was feared that

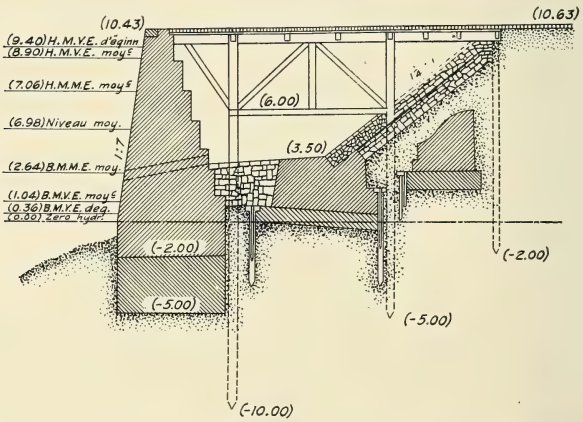

FIG. 14.—OLD QUAY WALL AT BOULOGNE, FRANCE, BRACED WITH REINFORCED CONCRETE.

the dead weight of the superstructure in this case would be insufficient to withstand the heavy seas.

A reinforced concrete caisson was finally designed by the Paris Hennebique Office and constructed in a dry dock up to a height of 51 ft. When floated the submerged depth was about 11 ft. 6 ins. The caisson was then towed to a place already prepared near the shore where all the interior walls were completed. The lower cells were filled with sand as it was considered necessary to obtain a draught of about 26 ft. for safety in towing. Fig. 15 shows the caisson floated previous to its being sunk in its final

place. The next step was to tow the caisson some 22 miles. This was done against a heavy sea at the rate of about a mile an hour. The site had already been dredged to about 50 ft. below mean high water and the caisson was sunk by pumping water into some of the interior cells. The last operation was to complete the superstructure. All these steps were completed without delay and at the completion of the job the contractors thought it would be entirely possible to take out the ballast at any time and tow the caisson to another site.

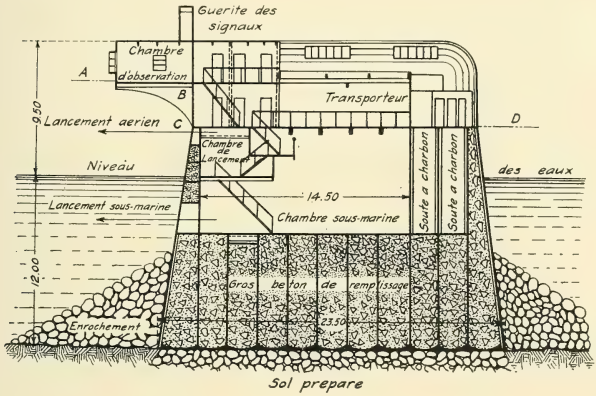

FIG. 15.—SECTION OF REINFORCED CONCRETE CAISSON.

These few examples of typical concrete structures show the wide range of adaptability of reinforced concrete. Most of the works described above were built between 1897 and 1903. Practically no special precaution was taken in selecting the sand, stone, Portland cement; it was, however, considered very important to use rich mixtures, these being generally 1 cement, 3 sand; or 1 cement, 2 sand or 3 gravel; or 1 cement, 1½ sand, 3 gravel—the proportion between the fine and the coarse aggregate being dependent on the percentage of voids.

In general the works described have withstood perfectly the ten or twelve years of action of the sea and do not show any

destructive effects, except where concrete is porous or the steel reinforcement is too near the surface. At Dundee, where the winters are very severe and the tides high, the reinforced concrete piers are in perfect condition; this seems to show that even the combined action of frost, sea water and pounding of waves have no ill effect on well constructed reinforced concrete works. It must be recalled, however, that in most of the works herein described such as pile work or caisson work, the concrete was

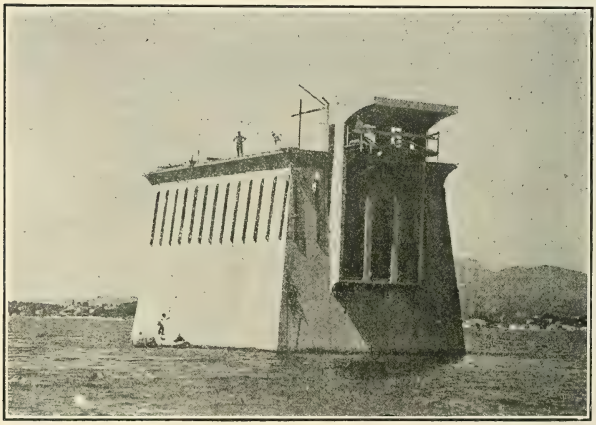

FIG. 16.—REINFORCED CONCRETE CAISSON LAUNCHED.

allowed to harden in air and was not subjected to the sea action before it had matured at least 30 days.

This paper would be incomplete if we made no mention of the works which have not been quite successful.

At the Charlestown, Mass., Navy Yard, a quay wall built of mass concrete shows signs of disintegration. The effect is localized between the low and high water marks and takes place after a severe frost. Authorities in charge of the work place the blame to the porousness of the concrete, as the water penetrates the pores and by freezing expands and causes the concrete to scale off.

At Hussum, Germany, a sea wall built in 1905 also shows

signs of disintegration. The mixture here was rather rich, but the concrete was deposited dry. Soon the surface subjected to sea water was covered with a white scum; then cracks appeared and later parts scaled off exposing the reinforcement.

Similar ill effects have taken place at a few other points. However, it seems that usually the concrete was found to be porous. Further action has been stopped by covering the surface with a waterproof material, such as cement finish, coal tar prep-

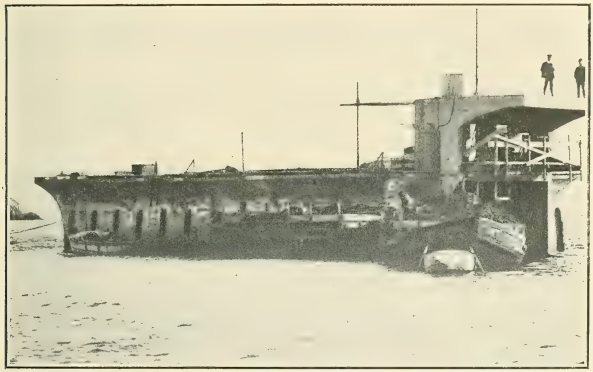

FIG. 17.—CAISSON SUNK IN PLACE AND READY FOR SUPERSTRUCTURE.

arations, Kessler Fluats, etc. It may also be mentioned that in some places, in Germany especially, trass has been used, the mixture being ½ trass, 1 cement, 3 sand, 4 gravel, but the addition of trass retards the setting of the cement.

Notwithstanding the poor results of some reinforced concrete construction in sea water in this country—in spite of the uncertainty whether we obtain a concrete always immune under any physical conditions from the action of sea water salts, and last, but not least, in spite of the prejudice which our leading engineers have against any new type of construction, millions of dollars have been invested in important sea water construction, not only by private interests but also by the War and Navy Depart-

ments of the United States, England, France, Germany, Denmark, Sweden and Russia.

The few works described give only a limited idea of the great strides made by reinforced concrete in sea water works. It can be stated that in this field of engineering reinforced concrete will prove, as it has with buildings, bridges, etc., to be the most permanent and economical material.

DISCUSSION.

Mr. Wig.

Mr. Rudolph J. Wig.—At the Atlantic City Laboratory of the United States Bureau of Standards (formerly under the United States Geological Survey) there are being carried on some rather extensive investigations of the behavior of concrete in sea water. Two series of tests were made, one of which might be called laboratory tests and the other practical tests. The laboratory tests were made in several ways. It was endeavored to determine the chemical change which takes place when cement is exposed to sea water, and a great many cements in briquette form were tested, giving very poor results. That is, cement exposed in the small specimens failed. In the concrete or what are termed practical tests, cylindrical specimens, 8×16 ins. were used, of various concretes, using several cements, and so far all specimens have tested well, with a few exceptions. Various methods of placing concrete in sea water were tried, such as placing through sea water, dumping from a bucket and placing through a tremie. Concrete specimens were also aged for various periods before placing in sea water. It was found that concrete could be placed in sea water immediately after placing in the mold, that is, filling the mold with concrete and immersing it immediately, and in time it would attain the same strength as though it had been allowed to set and age for a period before immersing.

The difficulty and effect of dumping concrete with a bucket from the top into the form is this: The cement, sand and stone are mechanically separated by gravity in settling, causing the materials to be deposited in layers, which result, of course, is similarly obtained in fresh water, but as an additional effect of the sea water its magnesia is precipitated and deposited between the particles of stone and sand, destroying all bonding value of the cement. In dumping from a bucket brought to the bottom of the form, the mechanical separation is almost entirely overcome, but over the exposed surface of each bucketful after emptying there will be a deposit of magnesia which affects the bond between this and the succeeding bucketful.

Mr. Wig. By depositing the concrete through a tremie or pipe more satisfactory results were obtained. When the molds were removed from the test piece made in this way, the concrete presented a fine looking surface, but upon testing would develop only from ⅓ to ¼ the strength of similar concrete placed in dry forms, even though in the latter case the sea water was permitted to cover the concrete immediately after it was placed.

Nearly all cements failed when molded in briquette form and exposed to sea water for from 13 weeks to a year.

It is not possible to reconcile the two series of tests, the neat briquettes with the concretes, but it seems probable that there is a change of structure with only a slight change of composition and volume, which in a small dense specimen, as a neat briquette, destroys its cohesive character. In concrete only a small amount of cement is exposed proportional to the total surface of concrete, and the sea water has to enter the voids or pores to come in contact with the cement when the magnesia of the sea water would be precipitated and assist in plugging up these pores, thus preventing the sea water from penetrating the interior rapidly. In a briquette there is a large surface exposed relative to its volume. If the briquette is exposed to sea water in a tank where there is no motion of the water, in some cases we find it will be heavily coated with magnesia and resist the action of sea water for a long period, but a similar briquette exposed in the sea where there is constant motion will not become coated and will disintegrate in a comparatively short time. No specimens have been exposed to sea water for more than 2 years.

Some tests are being made in which metal is embedded in the concrete to different depths and in different forms. These tests have been under way for about a year, and in most cases it is found that although the metal may not be covered by more than a ¼ in. of mortar and the sea water may have penetrated entirely through the concrete, the metal is in perfect preservation.

These tests covered several thousand specimens, and the work, as far as possible, was done in a practical way, endeavoring to eliminate the laboratory. Only the preparing or molding and the crushing or breaking of test pieces were undertaken in the laboratory. The exposure was made actually in the sea and not in

the laboratory tank. Upon breaking the concrete test pieces mag- Mr. Wig.
nesia could often be seen in the voids or porous sections as a white creamy coating. At the surface it would often solidify in a hard brittle yellowish scale. On analyzing the briquettes in some cases the analysis showed practically no difference between the one exposed to fresh water and the one exposed to sea water, and yet the sea water exposed specimen would have no strength while the fresh water exposed specimen exhibited normal strength. So that there seems to be a question as to obtaining an explanation by chemical analysis for the reason of the failure of the briquettes. Failure, I believe, will take place in some cases inside of 13 weeks. So it would seem as though it were a change of structure or the forming of some other compound in the cement without much change relatively in the elements composing the cement.

Mr. P. H. Bates.—Being a chemist, I am more interested in Mr. Bates.
what may take place chemically than otherwise. However, we hear that magnesium and sodium sulphate are alone the cause of disintegration of concrete. This is not entirely true. Magnesium chloride will decompose cement much more rapidly than magnesium sulphate, but while it may decompose cement, it will not disintegrate concrete. There is a difference between these two. For instance, it is possible to disintegrate concrete or mortars with plain sodium chloride. If you get sufficient sodium chloride, or any salt that will crystallize in the mortar, it will break it to pieces. But if you take a mixture of sodium chloride and sodium sulphate, for instance, you can decompose cement more rapidly than you can with sodium sulphate alone, but it will not disintegrate concrete more rapidly. The crystals of a definite weight of sodium sulphate will occupy a greater volume than an equal weight of equal parts sodium chloride and sodium sulphate.

Very much has been said about sodium sulphate and magnesium sulphate, or in fact any sulphate, combining with cement and forming with the alumina and lime there present, calcium sulpho aluminate. We hear constantly of Michaelis, the German authority, and LeChatelier, the French authority, having isolated this compound. However, neither did this, and both call attention to this fact in their works. It is due to Deval, another Frenchman, that this material has been isolated and studied, and who, moreover, first

Mr. Bates. called attention to it. By treating cement with a solution of sulphates and stopping at a certain time, when he thought the sulphate was no longer absorbed by the cement, he obtained a compound which closely analyzed to calcium sulpho aluminate. However, later he continued the action of the sulphate upon the cement, and was able to obtain compounds of an indefinite composition, depending upon the length of time the cement was treated. In the same way he formed a similar compound with chlorides, but was not able to isolate it.

To explain the disintegration of concrete by sulphates it is not necessary to assume that there is any chemical action whatsoever. You can take a material like a brick, or a sand stone, and saturate it with sodium sulphate repeatedly, allowing the sodium sulphate to crystallize out, and then it will go to pieces completely. This is due to the fact that sodium sulphate is a material that effloresces, that is, in a dry atmosphere loses its water crystallization and becomes a powder. Take a crystal of sodium sulphate and put it out in the air, and in a very short time it will become a powder, occupying apparently a much larger volume than previously, but actually a volume no greater than before and possibly less. But there is developed during this efflorescence considerable force which tends to disrupt the concrete.

There was a discussion of this entire subject before the American Society of Civil Engineers about a year ago, and in that paper was quoted the bulletin of Professor Headdon, of the Agricultural College of Colorado. He had submitted to him a disintegrated mortar tile, and investigated the subject by obtaining samples of the water in which it had been placed, samples of the original cement and sand, and also the disintegrated tile. From the analyses of these he shows that much magnesium sulphate was present in the disintegrated tile, and that there was an apparent loss in silica and alumina. However, he calls attention to the fact in his bulletin that this loss is only apparent. This is due to the fact that the addition of the magnesium sulphate, and the consequent replacing of some of the lime of the tile had decreased the percentages of silica and alumina. However, if calculated to a magnesia and sulphuric anhydride basis equivalent to that of the original cement, it will be found that the silica and alumina have

been decreased, but by a very small amount. In other words, while the percentages of silica and alumina in the disintegrated tile are less, yet the actual amount has not been decreased. **Mr. Bates.**

The statement that silica and alumina are removed from concrete during its disintegration by sulphate solutions is wrong. There is but a very minute quantity of either of these removed. In the case, however, of the so-called black alkali, which is sodium carbonate, both of these constituents are removed to a considerable extent, but in this case the lime contents remain constant.

Mr. H. S. Spackman.—I would like to ask Mr. Wig whether the briquettes were weighed. If Mr. Bates' idea is correct there should be a marked increase in weight of the disintegrated briquette or test piece. If the destruction is due entirely to the infiltration and incrustation of another material in the briquette it should be indicated by an increase in weight of the test piece. **Mr. Spackman**

Mr. Wig.—No, we did not weigh any of the briquettes. I do not think the weighing of the briquette would show very much. It was found that in some cases the increase in volume was very marked. The increase of the ordinary briquette in some cases was as much as ¼ to ⅜ of an inch on immersion in sea water, whereas in fresh water the increase would be very slight. The reason I do not think weighing would show anything is this: Some briquettes, even though they are totally immersed in sea water, seem to plug up and relatively dry out in the center. After the first 4 weeks of immersion a briquette will become saturated, after 13 weeks it will dry out in the center somewhat and after 26 weeks will be drier, which is a peculiar situation, but we have found it to be a fact in some cases. **Mr. Wig.**

I think the investigations of European laboratories have to a considerable extent been responsible for the general disbelief in the stability of concrete in sea water. Most of their investigations are made on these small specimens, and they have been drawing their conclusions from the results of tests on these small test pieces.

The President.—The Chair would like to say just briefly that during a trip to Europe he made personal inspection of the behavior of concrete in sea water and visited the sea water laboratories where a study of this subject was being made, and especially the laboratory of the Ponts et Chaussées at Boulogne sur Mer, **The President.**

The President. where a block of concrete had been in sea water for more than 25 years. And as a result of that study and observation, with the knowledge of all the tests made by Mr. Wig, which were inaugurated under the speaker's direction, the speaker feels that reinforced concrete construction is certainly adaptable for use in sea water, and that in the use of this material for that purpose, as in the use of the material for any purpose, it requires a special knowledge of the conditions and treatment of the material to meet those conditions. Like in many cases we find in alkali soils, where it is subjected to salts of various kinds, density is a prime necessity, and the harder the material is before it is subjected to such action the better it will resist the action. The conditions under which the blocks of concrete are exposed to the water has a material effect, of course, on the action of the water.

The blocks of concrete at Boulogne sur Mer were in the neighborhood of about 6-ft. cubes, and it was a rather porous, badly mixed concrete—mixed dry. The penetration of the water under the action of the tides resulted in a disintegration of the mass. There were cases in docks where the boats lying alongside the dock discharging and loading cargoes had abraded the concrete, and there were cases where the concrete manifestly showed, as at the Government station on the north coast of Germany, where the engineers in charge admitted that the concrete was made very dry, that the disintegration was due to the bad condition of the concrete. There is no doubt in the speaker's mind that concrete will continue to be used for sea water work. I think it behooves the engineer who uses it to understand the conditions and prepare the concrete in such a way as to properly resist the conditions. In some cases the speaker inspected there were cracks due to expansion and contraction from temperature. There again the design was defective. The engineer had not used reinforcement so as to so distribute the cracks and prevent the percolation of water.

SOME SPECIAL USES OF CONCRETE IN MINING.*

By George S. Rice.†

Concrete is not unknown to miners; apart from its uses in foundations for machinery, it has been employed for many years to plaster mine stoppings and dams built of brick or stone, to render them comparatively water or gas tight. Within the last few years, contemporaneously with the advance of the science of reinforcing concrete, the general use of concrete in mining operations has been enormously extended. To-day, practically all foundations for mine buildings and machinery are built of concrete, and many buildings, coal-bins, lofty chimneys, and even a large anthracite breaker have been built of reinforced concrete. In many of the large new mining plants, concrete has entered extensively into construction underground, replacing wood, iron, brick and stone. Its use for various purposes in and about the mines is rapidly extending, as its merits are better understood. The possibilities of underground application appear to widen constantly, suggesting more than mere economies and even affecting fundamentals of mining.

In the short compass of this paper, the present numerous applications and future probabilities of mine uses can only be enumerated and briefly touched upon in a suggestive way.

Shaft Sinking.

In Europe, mining operations have been regarded as of more or less permanent character, particularly in coal mining, owing to the great number of seams in the coal measures. Hence for many years shafts have been sunk using brick or stone for lining, such shafts being circular or elliptical in cross-section. Where it has been necessary to employ iron or steel casings to pass through water-bearing strata, permanent inner linings of stone or brick have been used. In some recent shafts concrete has been substituted.

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In this country until the past few years, it has been without exception the practice to employ wood linings, the shafts being rectangular in cross-section. Water-bearing silt or quick-sand locations were generally avoided, and where water was encountered in shaft sinking, no attempt was made to keep it out as in the European shafts.

In some instances where the surface quick-sand proved shallow, concrete dams were constructed inside of timber lining or

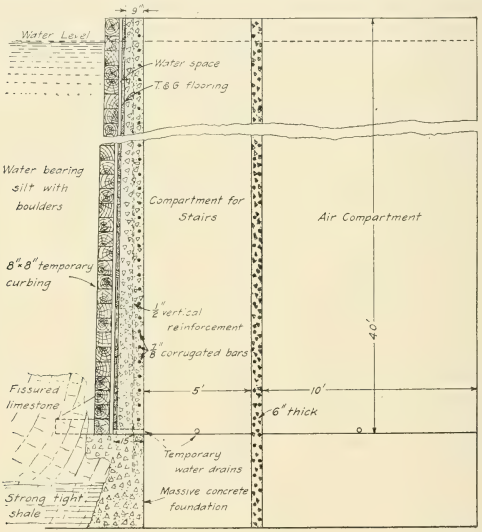

FIG. 1.—SKETCH OF RECTANGULAR REINFORCED CONCRETE SHAFT-DAM TO WITHSTAND WATER AND GROUND PRESSURE.

coffer dams. Generally these inner linings or dams were circular or elliptical in section and massive so that reinforcement was incidental.

Example of a Reinforced Concrete Lining.—In sinking the air shaft of a mine in Illinois, in 1906, the writer used a reinforced concrete lining, Fig. 1, of rectangular section through the surface silt of sufficient strength to resist the water pressure, so

as to keep the air-shaft dry. This is an important matter if the shaft is to be used for "intaking" air, since in a wet shaft, masses of ice accumulate in cold weather, thus restricting the ventilating current or stopping it with serious results. The inner lining was put in after the shaft had been forced through the soft silt, and the outer wood curbing rested on this rock. The latter was a fissured limestone so the shaft was sunk about 8 ft. further into the shales below—here a good foundation was prepared for the concrete lining.

As the water rained down the shaft in streams, while putting in the massive concrete foundation, the sinking pump was temporarily checked in laying the successive courses, and the water allowed to rise above the level of the course to be laid, so that the concreting materials could be placed behind the form in water without currents. The concrete was then carefully rammed against the shale and rock to make tight joints.

When this foundation had been brought up to the level of the wood curbing, preparations were made for building up the thinner reinforced concrete walls, which were but 15 ins. thick at the bottom, and 9 ins. thick at the top, 40 ft. above. Diagonal wood strips were nailed on the inner face of the wood curbing and to these a tight lining of tongued and grooved flooring was nailed. This deflected the water streams to the bottom of the lining and short pipes were placed through the concrete forms at a number of points around the shaft so as to discharge the water from behind the wood lining into the open shaft. By this method, the reinforced walls were erected with freedom from running water.

The reinforcement consisted of deformed bars placed horizontally and vertically near the inner face (*i. e.,* toward the center of shaft). The vertical bars (½ in. square) served only to bond the courses—the horizontal bars (⅞ in. square) were spaced close together at the bottom, and further apart at the top, according to calculated water and earth pressure, assuming no help from the outer wood curbing. The horizontal reinforcement bars were bent around at the corners of the shaft, which were beveled. The shaft had two compartments about 9 x 10 ft. and 9 x 5 ft. in the clear, with a 6 in. concrete partition between.

When the concrete had sufficiently hardened, the drainage

pipes were plugged, and the walls then served their object as a dam. There was a slight seepage at several points but this gradually stopped.

DROP SHAFT SINKING.

Where the glacial drift or water-bearing silt is very deep, special methods of shaft sinking have to be employed. The freezing system and the drop shaft system have been highly developed in Germany and France, and to a point where both are very successful. Reinforced concrete is adaptable for linings in connection with either system, but it has not as yet been employed to any considerable extent.

In this country, little has been done with either of these systems, chiefly because there has been no general need as shaft sinking conditions are ordinarily easy. More recently there has been occasion for penetration of water-bearing silts in the Lake Superior iron mines, and one reinforced concrete casing has been successfully sunk. The plan of the casing was similar to the ordinary drop shaft casing of brick so extensively employed in England, namely, to build on top of a steel shoe which drops as the excavation proceeds. Skin resistance to descent of the "drop shaft" is the chief obstacle usually encountered so far as the casing is concerned, and this has to be overcome by weighting.

The concrete bridge pier caissons sunk in western river silts, and the caissons used in the vicinity of New York for shafts preliminary to tunneling, are of the same nature as drop shaft casings, except their vastly greater size and weight, relative to depth, makes the problem of sinking them simpler. The small caissons sunk for foundations of high buildings are more akin to mine shaft sinking, except the depth is usually less, so air pressure can be used to hold back the water.

Drop shaft casings can be sunk by dredging with clam shell or orange peel buckets under water when the material is silty or sandy. If complicated by glacial boulders bedded in hard clay or natural cemented material, it is necessary, when the work is too deep for air pressure, to pump out the water and excavate by hand with the assistance of explosives.

The method of using reinforced concrete casings seems applicable to many situations where it is necessary to sink through bad ground, modifications being made to suit the condition of the sand or glacial drift that may be encountered.

Cementation of Wet Ground.

A most interesting method used in connection with shaft sinking or tunneling through water-bearing ground has been developed in the Pas-de-Calais coal district of France, which is termed "cementation." The coal measures of the district are overlain with bedded marls and chalks, more or less fissured, and the fissures in some places are so filled with water that ordinary methods of shaft sinking would fail. Heretofore, the freezing system has been successfully used, but the new cementation method, the writer was informed by French officials who had used both methods, could be employed at less cost.

In one instance, the cost of cementation through 343 ft. of water-bearing ground was $13,200.00. Interesting particulars of this work are given in a paper* by J. Lombois, principal engineer of Mines de Bethune.

Briefly, the process used by this company is to drill a set of holes in the water-bearing ground, six in number, arranged in a circle, surrounding the site of the proposed shaft, and then to pump a cement grouting through the holes until the fissures and interstices are filled. Shaft sinking is then proceeded with as it would be through ordinary rock strata. The system seems applicable to many situations where water is encountered in either shaft sinking or tunnel driving in broken rock.

Shaft Landings or Bottoms.

In Europe it has long been the custom in collieries to secure the "landings" or "bottoms" adjacent to shafts with masonry side walls and arched roofs, both for fire protection and for permanence. Such arched passages or tunnels contain from two to four tracks, and where double-decked cages are employed, are sometimes arranged with corresponding upper and lower stages requiring considerable height of tunnel.

Bulletin de la Societe de L'Industrie Minerale, Vol. IX, 1908.

In this country, until recently, with rare exceptions, the shaft bottoms have been lined with timber only. Such mine timbers, if of white oak, last from 5 to 8 years, depending on whether the natural conditions foster dry rot, and if of black oak or white pine, shorter periods. As the cost of replacing such timbers is greater than their first cost, when a mine lasts over 20 years it is manifest that wood timbering is in the long run more expensive, and there is always the great danger of its taking fire.

Where the bottom is to be arched, brick is an excellent material to use, and the substitution of concrete is wholly a question of relative cost. There are, however, many situations where there is a strong cap-rock over the coal that is difficult and expensive to brush in making room for the arching; moreover, such cap-rock may have considerable strength as a roof if protected from weathering. Under these conditions, the practice has arisen in recent years to use large (15 to 18 in.) I-beams for collars which cross the passage, and the ends resting on side walls, and to lag with wood over the beams. Such a construction makes a good appearance and should be fairly durable, but it is open to some objections. The wood lagging will rot and be difficult to replace without disturbing the roof and, in case of an explosion, the beams offering considerable surface, may be blown down or, if struck by flying timber, be broken. This has happened in several recent mine explosions. Moreover, the side walls not being held apart, are likely to be blown into the tunnel by the exploding gases getting behind the walls or by the suction that follows the cooling of the gases. Further, in case of an extensive mine fire which may be sustained through the "bottom" by the burning of cars of coal, the I-beams would be seriously affected unless protected by fire-proofing.

There appears to be an excellent field for the application of reinforced concrete under the conditions cited; namely, where a flat roof and smooth sides are desired of durable and fire-proof construction. Investigations will have to be made to determine what should be the strength of the flat concrete arch acting as a beam, under different conditions of natural roof, and in the development of methods of erection under horizontal roof stratum. Such testing and experimenting might be an excellent field of

investigation by the newly created United States Bureau of Mines at their experimental mine near Bruceton, Pa., and elsewhere.

Lining Entries with Reinforced Concrete.

The roof stratum over most coal seams is shale, which when hard, is called slate by the miners. Shale generally weathers badly and in a majority of coal mines, the entries must be timbered more or less continuously. If the shale were protected from the weathering action caused by the ventilating currents, it would require little support.

This principle has been recognized by the Bethune Company which operated collieries in the Pas-de-Calais district of France, so that in developing a new mine a few years ago, they decided to line all the passageways with reinforced concrete. At the time of the visit of the writer in September, 1908, about two miles of such lining had been constructed, and while of light construction, it had yielded slightly at only one point.

Fig. 2 gives the cross-section of a large passageway lined with reinforced concrete and shows the forms in place. The side walls and arch are uniform in thickness except for irregularities of excavation. The minimum thickness is 6 ins. (15 cm.). Two cross-sections of passage are employed, the main galleries 9.5 ft. wide in the clear, and 8.5 ft. in height (290 x 260 cm.). The secondary passageways are 7.25 ft. wide in the clear and 7.25 ft. high (220 x 220 cm.).

The reinforcement consists of arched rods 0.4 ins. (1 cm.) square, placed 32 ins. (80 cm.) apart. Longitudinal reinforcement is furnished by round rods 0.2 ins. (0.5 cm.) diameter, placed 9 ins. (23 cm.) apart. The arched reinforcement is placed 3 ins. (75 mm.) from the form.

The forms are ingeniously constructed so as to be readily transported and set up quickly, Fig. 3. The same gang of men are steadily employed on the work and become very proficient.

The concrete material is a mixture of boiler ashes and burned shale with cement, prepared with a little excess water and mixed in the mine by hand, or laterly by a mechanical mixer. While the time of setting of the concrete is variable it usually does not exceed 15 days.

When the rock passed through is faulty it is sometimes advisable to inject a cement grouting to fill the voids behind the lining. This can be cheaply and expeditiously done by using compressed air operating on a cylinder filled with grout.

The average cost for the small passage is about $5.00 per linear yard. The small passageway is narrower than could be used in American collieries, which employ large cars. For American practice a minimum size of 7½ to 8 ft. would be necessary for single track roads; in addition to which refuge holes would be required by various state laws respectively at 40 to 100 ft. inter-

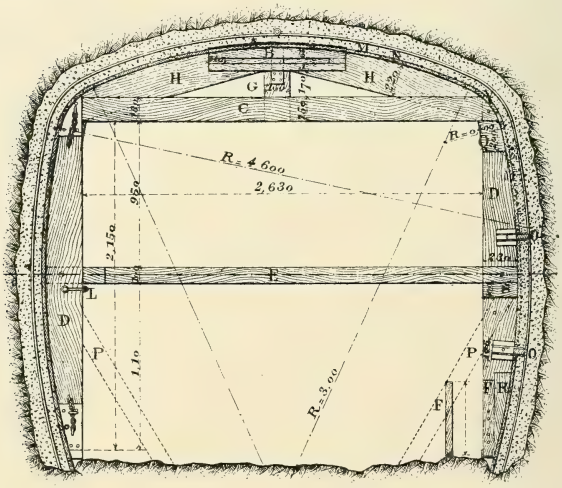

FIG. 2.—CROSS-SECTION OF PASSAGE WAY LINED WITH REINFORCED CONCRETE WITH FORMS IN PLACE.

vals. For the mines of this country, it is probable that in using wider sections and requisite refuge holes, with the higher labor cost prevailing, the cost of such concrete lining would approach $8.00 or $10.00 per linear yard of entry. While at first sight this cost seems prohibitive, and is so for temporary entries, undoubtedly it would be an economy for such passageways as may be in use over 9 or 10 years, and which due to natural condi-

tions would require the expense of retimbering every 4 or 5 years, in addition to the cost of the loading out of large falls from time to time.

In the case of the Bethune Company, their management informed the writer that the "cost of maintenance"* of the mine in which the lining was applied was only one-fifth of the average cost of maintenance of the other mines of the company operating with the same concession (government lease).

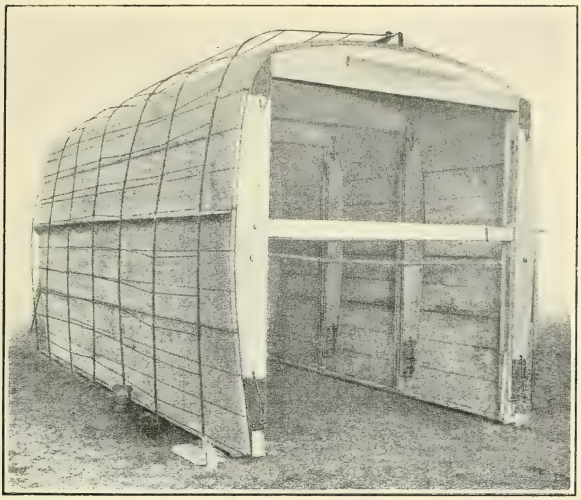

FIG. 3.—SECTIONAL FORM WITH REINFORCEMENT IN PLACE.

Other advantages of such lined passageways are manifest to a mining man. There is no blockage of traffic by falls; watering the roads can be done freely and little is required to keep the passage free from dry coal dust, that great menace to miners and the cause of propagation of so many great explosions. Moreover, with refuge holes, there is less danger of employees being injured by passing trips of cars. In timbered entries, if cars jump the

*"Cost of maintenance" in this case means the replacement of timbering and handling rock from falls along the main roads.

track, they are liable to knock out the timbers and bring down the roof on either the trip rider or a man standing at one side. Good lighting tends to reduce haulageway accidents and a concrete lined passage is easily lighted since the surface is smooth and light-colored—and if desired it can be cheaply whitewashed.

The apparatus for applying cement mortar by means of compressed air,* commonly known as the "Cement Gun," offers great possibilities for the lining of passageways, etc., with concrete. By protecting from weathering the roof and walls of a passageway with a thin coating of cement, it is possible that the heavy expense of timbering in many cases may be avoided. The machine also offers possibilities of use in the fireproofing of timbering board stoppings and in the erection of firewalls in places difficult of access, since the material can be pumped for a considerable distance.

OVERCASTS, STOPPINGS AND DAMS.

Concrete has already been adopted by many mine operators in the building of overcasts (air current crossings). These have usually been of massive construction. If reinforcement were employed, the cost of overcasts, also of stoppings and dams, might be considerably reduced. Some operators use small coal for stoppings in place of crushed stone. If the coal is of a tough nature and properly screened, in massives stoppings this is economical, but often too much coal dust is used so that an excess of cement is required and the concrete is very weak. Such concrete is not suited for stoppings or water dams where there is likely to be much pressure thrown upon them. It is also not suitable for fire stoppings for obvious reasons.

CONCRETE PROPS.

There is undoubtedly a large field for the development of a system of concrete props, molded and hardened on the outside of the mine. Props are a large item of expense in most collieries. In the average colliery in this country, one room prop is required for every 8 to 12 tons of coal produced, say 1 to 10.

*See paper on "The Use of Compressed Air in the Handling of Mortar and Concrete," pp. 504-521.—ED.

Timber props cost from 10 to 20 cts. each, so the cost per ton of coal produced is from 1 to 2 cts. The cost will undoubtedly rise rapidly in the future as the timber reserves of the country become exhausted.

In this country, timber props are rarely recovered, as they are abroad where the longwall method of mining is general. In the room and pillar system which is the one usually employed in this country, the props often stand so long as to dry rot and not make it worth while to try to withdraw them in pulling pillars. In collieries in which the conditions are such that the props could be withdrawn by "prop-drawers," such as are used extensively in England, the use of concrete props might be a great economy, besides conserving timber resources.

Concrete Timber Sets.

There appears to be an even better opportunity for the development of a system of outside constructed reinforced timbers for passageways than there is for room props, since timber sets are intended for permanent use and greater strength is required. In collieries, each timber set consists of a collar and two posts or legs. The collar is preferably square in cross-section; the legs may be either round or square. It would be necessary to have legs of different lengths, but this entails no particular difficulty; small differences are easily taken care of by picking the footing deeper, or wedging over the top of the collar. That there is a field for the use of concrete timbers is shown by the increasing use of I-beams for collars, and for legs. Concrete timbers for equal strength would be more durable and should be cheaper.

The opportunity for use of concrete posts and timber sets in certain kinds of metalliferous mining appears to be as great as in collieries. Often great strength is needed, and in some parts of the country as in the southwest mining districts, big timbers are difficult to obtain and very expensive. Crushed rock for the concrete is nearly always at hand in metal mining operations, and Portland cement can now be obtained at moderate price in any part of the United States. Reinforcing material is often found in the scrap iron pile. In some cases, the large concrete timbers could be built in place, but in many instances, owing to the condition of the roof, it would be necessary to have the full

strength of a concrete timber at the time of erection; hence the necessity of having concrete timbers constructed and seasoned in advance of setting in place.

CONCRETE TRACK TIES.

Among other mine supplies subject to rapid deterioration, there is the considerable item of track ties. Frequently there are miles of tracks in a mine, especially in a large colliery, for which an immense number of ties are required, running into the tens of thousands in a single mine.

Ties have a shorter life than props or timbers, inasmuch as they are subject to both wear and rot. In most mines the floor of the roadway is generally damp, and in the case of bituminous mines it is purposely wetted in order to lay the dust. This damp condition favors the rotting of ties. Ties as a rule are made of oak, preferably white oak, and are either split or hewn like railway ties. They have to be renewed at intervals of 2 to 4 years. Here then is an opportunity for the concrete tie if it can be adapted to the conditions in the main or permanent roads. Possibly if made light and durable enough, they could be used for temporary room ties and be taken up and relaid like the rails in another part of the mine when the room was finished.

In the development and testing of concrete props, ties, timbers, as well as the concrete linings previously mentioned, there would appear to be need of work on the part of the United States Bureau of Mines to undertake tests under practical conditions for the benefit of the mining industry.

MINE TOP WORKS AND HOUSES.

The use of concrete for outside mine buildings has been merely mentioned in this paper, as such work is either identical or very similar to what concrete builders are thoroughly familiar with. But it is not intended to give the impression that the amount of this work is small or that it is unimportant. Every year there are heavy money losses from fires in tenant houses and mine structures, which, in the case of shaft houses, have sometimes caused the loss or temporary loss of the mine, involving indirect losses of thousands of dollars.

The aggregate of new construction for mining plants annually reaches large sums. In the case of large modern collieries, a single plant may involve, exclusive of machinery, for outside buildings and miners' houses from $100,000 to $200,000. Generally the construction of the mining plant is under the direction of a mining man and the buildings are treated as incidental to the installment of machinery and mine development, which is usually pushed with the utmost vigor. Under such circumstances, the construction of the general buildings is not given the thought and attention that they warrant, and buildings are often makeshifts. If concrete builders would give the subject the same study that they give to bridges and buildings for manufacturing, and be prepared to take up the construction of mine buildings of various kinds with promptness, there appears to be a large field of usefulness, productive of improved construction and adequate fire protection.

REPORT OF THE COMMITTEE ON REINFORCED CONCRETE AND BUILDING LAWS.

1. Tests on Reinforced Concrete Structures.

Proceeding along the lines suggested at the last convention, in order to secure data of value as to the actual carrying capacity or stresses in reinforced concrete structures designed according to current practice and particularly to throw some light on important questions relating to the design of such structural members at present more or less indeterminate theoretically, the Committee proposes a series of tests, the data from which, if secured under satisfactory conditions, will greatly assist the concrete user in taking every advantage of the material and still have a conservative design. It is suggested that a series of tests be arranged and carried out on buildings either during construction or after completion, so that the information may be obtained under conditions which will show the actual stress in structures under working loads.

Up to this time tests of completed structures have consisted almost exclusively of a determination of the deflections in relation to applied loads, although ocasionally a section of a building has been tested to destruction. Because of the indeterminated stresses in any structure consisting of a combination of members of different types or of continuous construction, involving the elasticity or variation in stiffness as well as arch action, such tests are of comparatively little value in evolving laws and formulas for design, by reason of the impossibility of duplicating conditions in different structures.

Recently, however, methods have been devised to obtain in a scientific way not only the deflection under different loads, but also measurements of deformation of the concrete and of the reinforcement from which can be calculated the stresses in different parts of a structure. A notable example of such a test is described in a paper presented at this convention.* A thorough and pains-

* A Test of a Flat Slab Floor in a Reinforced Concrete Building, by Arthur R. Lord, p. 180.—Ed.

taking analysis of tests along the same lines, if repeated on structures of varying design, will give results affording a basis of comparison with test data already available of individual members, such as single beams, slabs and columns.

To conduct such tests it is necessary to secure, first, the co-operation of the owner and architect or engineer, in order that the test may be made at all and second, the co-operation of men familiar with the making of accurate observations and necessary records for the proper analysis of the test data. As ordinary tests are frequently made, even when the design and execution of the work are under the control of competent building bureaus, the expense of the proposed tests will be insignificant compared with the cost of especially constructed test panels.

The Committee offers every assistance wherever such tests may be made by arranging for the necessary apparatus and the proper men to make the observations. The Committee hopes to present in its next report analysis of such tests on several buildings.

2. Uniform Building Laws.

The Committee realizes the desirability of uniform building laws in this country. The primary function of a building law is to insure public safety, *i.e.*, to define as specifically and as clearly as possible the limits in the design and methods of erection of structures, so that within the range of conditions and the uses for which the structure is designed, a certain margin of safety against failure may exist. The natural tendency of the individual is always to build his structure at the absolute minimum of cost, so that instead of following what may be termed good practice, full advantage is generally taken of the limits of the law whenever the cost of the structure can be so reduced. Consequently, the drafting of a law involves the old question of the rights of the individual in relation to the rights of the community.

In view of the importance of the element of cost, an attempt has been made to trace the influence of the more important requirements of typical codes governing reinforced concrete building design. Unfortunately, so much labor is involved in obtaining these comparative costs that only one type of building is being considered. The Committee is working out a design and estimate

for a typical building, selected at random, under the building laws of the principal cities in this country, as well as under the building regulations of this Association and the requirements of the Joint Committee on Concrete and Reinforced Concrete. The proper preparation of a design and estimate of cost for a typical building under the building laws of a city requires the services of a competent designing engineer who is thoroughly familiar with the requirements in that city. Inasmuch as the Committee must await the pleasure of such competent engineers who have offered to prepare the data for their city, the complete report is not available now, but will form part of the report of the Committee for next year.

Respectfully submitted on behalf of the Committee:

ALFRED E. LINDAU,
Chairman.

TOPICAL DISCUSSION ON CONCRETE AND REINFORCED CONCRETE.

Corrugated Iron Floor Forms.

Mr. Leonard C. Wason.—One new method in construction work is the use of No. 20 gauge black corrugated iron for floor forms. It is not original with me, but first came to my notice last February in Minneapolis where it has been used quite extensively. It was developed out there because a sheet metal worker and roofer became interested in this type of work, furnished the metal, erected and took care of it, straightened it and so forth, at a very reasonable charge. No metal worker would do this in Boston, so we undertook it. The results were very satisfactory in the way of cost, and as it was used in flat floor construction there was very little fitting to do. **Mr. Wason.**

The junctions of column heads with the floors were made with a square metal plate fastened to column head. The inside was the outline of the column, sometimes octagonal and sometimes circular, but the outer edge was a square. The corrugated plates lapped on to the square and by sliding them in or out longitudinally the right amount of lap and closure of floor form was obtained. Lapping over one or more corrugations crosswise gave the correct fit the other way, so there was practically no cutting to do. The forms were supported by planks on edge, 24 ins. apart, and did not sag when loaded.

Of course if the appearance of the ceiling is objectionable this type of form could not be used; but in a storage warehouse the appearance is of no particular consequence. Although there were some laps giving a rather blunt edge, the finished appearance was no worse than frequently obtained from poor joints with boards.

In regard to economy, the corrugated iron forms cost about one-half as much as wooden floor forms which I have used for a good while, and from that standpoint were a great success.

When drawing the false work nearly all the props were taken out leaving just barely enough to hold the forms up. Then ropes

Mr. Wason. were tied around the remaining props and with the men at a distance where there was no danger of injury, the remaining posts were all pulled out at once, dropping the forms of a large room at a single operation. The corrugated iron came down pretty straight, not being bent up much, and two men with a hand straightener cared for what little damage there was. A steel die about a foot square was used, the shape of the corrugations, and another one was supported above by a very crude set of levers which would drop into place, then the sheet was hammered into shape with a heavy maul such as is used for driving sheet piling. Two men were able to keep up with the straightening of sheets where 60 carpenters, with about 30 laborers attending them, were used for erection.

After the forms had been used about 8 times they were practically as good as new except for a few nail holes, as they had to be tacked down at intervals along the sides and ends. These forms went through the first job so well that we expect to use them 8 times more. It has been our experience that if lumber is used 3 or 4 times we are doing very well indeed. In some special cases, where forms have been made up of heavy material they have been used more than that; but even then they are not as economical as these corrugated metal forms.

The iron forms are greased with crude oil to prevent sticking of the concrete and this gives rather a sleek and smooth appearance to the ceiling. There was very little roughness due to adhesion of the cement to the iron, and in a few cases where the grease did not prevent sticking, there was no necessity for patching. Of course, if there is much patching to do it might take a little longer than with a plane surface.

Mr. Lindau. MR. A. E. LINDAU.—Corrugated iron forms have been used in St. Louis quite extensively in the last two or three years, and the appearance of the underside of the slab is very satisfactory. Quite a uniform surface is obtained which looks fully as well and in some respects even better than the concrete in which timber forms have been used, particularly in connection with girderless floors.

CONCRETE AGGREGATES.

Mr. Thompson. MR. SANFORD E. THOMPSON.—The defects in concrete aggregates may be divided into two general classes. One class where the

material in its natural state is absolutely unfit to be used for concrete because of impurities; and another class where the material is clean but the size or gradation of sizes of the grains is such as to give a low strength of mortar or concrete without specially grading or an extra amount of cement. Mr. Thompson.

The first class, comprising materials which are absolutely unfit to use, is of the greatest importance from a practical standpoint because with this character of materials we are having every little while actual failures of concrete structures,—the concrete failing to harden so that the building or other structure falls when the form is removed. The use of such material as this is complicated by the fact that it frequently is extremely difficult to detect by the eye, even by the most careful examination, whether the sand contains impurities which unfit it for use.

It has always been the custom for the engineer or the architect or the contractor to take up a sample of sand in his hand, examine it, rub it in the palm of his hand, and accept or reject it on a basis of this inspection. Then if the concrete fails to harden, the fault is blamed upon the cement. However, the further one goes into investigations of sands, the more certain it becomes that it is an absolute impossibility to determine by inspection of a sand whether it is fit to be used in concrete. The difficulty is that the impurities, while affecting the chemical combination with the cement, may be so minute as to be impossible to distinguish by the eye. For example, an extremely small percentage of vegetable matter of certain kinds may delay the hardening of the cement so that at 7 days there is practically no strength in the concrete or mortar.

Engineers must come to a point where they will test sand just as carefully as they will test the cement. A recent case illustrates how easy it is to be deceived in the quality of sand. A lot of material—sand and gravel—was sent to my laboratory for the purpose of determining the best proportions for concrete. The sand looked all right and we went ahead and made the necessary mechanical analyses, but as a precaution also made tensile tests of the sand mixed with cement in 1:3 proportions. We found, notwithstanding the good appearance of the sand, that the tensile strength of 1 : 3 briquettes at the age of 7 days was about 40 lbs. per sq. in. instead of the 200 lbs. which is apt to be obtained with a good sand. Later on we made further tests of

Mr. Thompson. another sample of sand from the same pit, and after a very careful inspection and microscopic examination of the grains and a comparison with the previous sample, we decided that it was probably good sand, and as we needed it immediately for use in our investigations, sent for a large duplicate sample by express, without waiting for test. Now when we came to test this former sample, we found that it had just the same characteristics as the rest of the pit and gave scarcely any strength at 7 days.

Such examples as this, and they are occurring constantly, make it evident that the best looking sand must be tested before using. The simplest test is the ordinary one of mixing up the sand and cement into a mortar in proportions 1 : 3 and comparing the strength of the briquettes made in this way with briquettes of standard sand mortar in the same proportions, made at the same time, under the same conditions. The requirements of the Joint Committee on Concrete and Reinforced Concrete are that the sand in question must attain a tensile strength of at least 70 per cent. of the strength of the standard sand briquettes.

Sand which contains impurities can sometimes be used after special treatment. In some cases, it can be washed satisfactorily. By washing I do not mean turning the hose on the pile of sand, this is absolutely useless. But by constructing a special apparatus, such as a box containing perforated pipes through which water is forced under pressure, the silt rises to the surface and flows off with the surplus water, while the washed sand may be drawn off through a gate at the bottom.

In certain cases, it has been found that a change in the brand of cement will give better results with a certain sand. This seems very curious since Portland cement, as made by the different companies, is of such uniform composition and tests so nearly alike when used neat or with standard sand. However, several cases have occurred later where it was found that by using a different brand of cement, a poor sand would set up satisfactorily and make good concrete.

It is evident from the above that the sand question is a very broad one and the selection of sand must be made with a great deal of care and judgment.

Mr. Ham. MR. WILLIAM H. HAM.—An experience which I had with concrete this year showed the relation of the character of sand to the

strength of material. Six footings of a group of some 50 were cast and failed to set up. These were left in place before removal some 10 days—I think exactly ten days—and a portion of most every footing had set up while other portions of the same footings had not set up. Mr. Ham.

Investigation was made as to the cement, and it was found to be of standard quality. The sand, as far as could be seen was satisfactory, with the exception that it had been taken from the bank with an orange peel bucket, and the special grade used in the footings had been dipped through the water. The footings were taken out and replaced, and I took two samples of the concrete which could be easily broken into pieces after ageing 10 days. These samples were pulverized and re-mixed with water and this concrete set up very solid.

A thorough investigation of the sand pit showed that just below the average level of the sand in the bank being used, was a strata of quicksand. This was detected by driving pipes and removing the cores. The presence of quicksand in small quantities is very dangerous in concrete work. The blocks made from pulverized poor concrete mixed with water ten days after original concreting, are to be tested for compression.

Some seven years ago I had charge of the preparation of sand for a filtration plant at Yonkers, N. Y., Mr. Allen Hazen being consulting engineer. The sand in the vicinity was not up to the standard required, and a washing plant was designed by Mr. Hazen and erected under my direction. About 6,000 yds. of sand were washed at a rate depending wholly upon the number of teams available for hauling the sand to the top of the charging platform. With about 20 teams, a maximum of 250 cu. yds. of sand were washed in 10 hours. The washing plant, Fig. 1, consisted of a receiving hopper at the top, two troughs, one feeding into the other, with a drop between the two and a screen inclined at an angle of 45 deg. at the ends of the troughs, the screen separating the different sized gravel as the material was washed down from the top. A centrifugal pumping plant raised the water 13 ft., which was the total drop; the water feeding at right angles into the bottom of the hopper, so as to shoot the sand into the flume. A by-pass from this pipe line connected into the washing

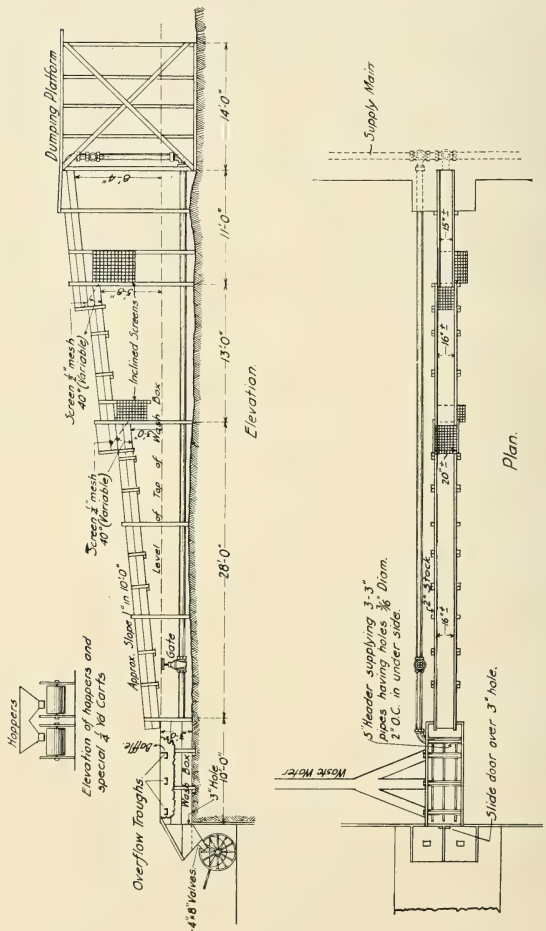

FIG. 1.—WASHING PLANT FOR SAND.

box which was located about 40 ft. from the hopper and on a lower level. Mr. Ham.

The washing box, 3 x 10 ft., contained a series of three pipes perforated with holes on the bottom and fed by a main pipe. While washing the sand was allowed to flow in at the top until about one-half full, then the water was turned on and kept on continuously, and the sand continuously flowed, the water running over the cross flumes at the top of the box, taking away the floating material. The sand was drawn out through a 3-in. hole at the bottom.

The centrifugal pumping plant worked against a 13-ft. head, with a 6-in. discharge and 8-in. suction pipe. In addition to the supply of water underneath the sand there was available a 2½-in. line from the city water pressure, so as to provide additional pressure if required. The sand and gravel washed by this process carried about 30 per cent. gravel above ¼ in. This was separated in two piles by screens about 15 ins. square and the sand carried through the lowest screen into the washer where the loam and clay were removed, showing a change in color from a dull brown to almost a white, like white sea sand.

It was estimated that from time to time 5 to 13 per cent. of fine material was removed. A lot of clay was taken out, too, and the river taking the effluent was yellow for three miles down the stream.

Mr. Charles D. Watson.—The sand washing plant described was probably a very good plant, but some of us have only small work to do and have to use a quick means of washing sand. Recently we used an inclined vibrating screen with a separator. The feeding of the screen is by means of a spiral groove which forces the sand over a screen plate. It is very easy to place a pipe at the point where the sand falls over, and force the water into the sand. This would be a sand screening and washing plant which can be arranged in a few hours, at a minimum cost, and it gives excellent results. Mr. Watson.

Mr. Arthur N. Talbot.—The question of proper design and execution of concrete construction is one in which I am much interested and it seems to me as I go further along in this field, that we need to-day a great deal more experimental work in order to get a satisfactory basis for our construction plans and in order Mr. Talbot.

Mr. Talbot. to explain some things in the way of variation of materials and differences in results and in methods of calculation in different types of construction.

Just at the close of the discussion of this last topic there came to my mind an illustration—some tests made on about twenty different sands that were in common use in as many cities in the State of Illinois. These sands were tested in a manner somewhat similar to that described by Mr. Thompson—by comparing the strength of mortar briquettes with the strength found with the same cement and standard sand—and these twenty sands varied from 40 to 115 per cent. in strength, as compared with the strength of the standard sand. And yet, they were in common use in various cities of the state. It is only another illustration of what has now become rather common knowledge, that we must not lay everything to the cement; that we must not expect that with good cement alone we shall get good work. We need to give attention to the aggregate which goes into the concrete.

INVESTIGATIONS.

MR. TALBOT.—*Footings.* Of the tests we have been making in the Laboratory of the University of Illinois along the line of reinforced concrete, I think perhaps the newest line is the test of footings. Footings have been built of reinforced concrete similar to those in buildings, and they have been loaded in such a way that the pressure over the base of the footing is fairly uniform—the same as we expect to get in soils.

The design of these footings has been such as to give in some cases failures in tension in the reinforcement; in others, shear failures, and in still others failures in bond resistance between the concrete and the reinforcement. I have been struck with the probability that in many footing designs, particularly with the larger rods used, a failure in the footing would come from a failure in bond between the rods and the concrete. That we do not have more failures reported I suppose is due not so much to the merits of the design as to the fact that after incipient failure, the pressures on the under side of the foundation will adjust themselves and instead of being uniformly distributed as they might be at first, the pressure becomes much larger in the center of the founda-

tion, and also that when settlement occurs it is laid to poor soil rather than to poor footing. Mr. Talbot.

In that connection I may say that the width of beam available in such a footing, is much wider than is generally given, or than I had supposed. Very nearly the full width of the footing is available as a resisting beam for the loads which come upon it instead of only the narrow width under the pedestal of the footing.

Slabs. We have made a test of slabs supported on four edges in an effort to find how the reinforcement is distributed from end to end.

Columns. Some tests were made which throw light on a question sometimes in controversy as to the value of longitudinal rods in columns; these tests go to show that the method of testing has a great deal to do with the strength obtained, and that the method of placing the rods in the column is of much importance. Columns were made with rods having a good square end bearing on the plates above and below, their ends being milled. Others had the ends of the rods merely sheared; others were built with the end of the rod not reaching the face of the plate by say 2 ins., while others were spliced in different ways.

It was found that those with square end bearing gave very high results, much higher than is ordinarily credited to such reinforcing by rods, the usual relations between stresses in concrete and reinforcement. Those with sheared ends gave results not so high, but still fairly high. Those in which the reinforcement did not reach the bottom, but was merely supported on a block or plug of wood, gave results not much higher than the plain concrete. Those with sleeve connection at the middle of the column, gave fairly high strength, but not as high as the continuous rod. It is plain that in a building of 8 or 10 stories the load is being applied from floor to floor as we go down the building, and under ordinary conditions, if the rods are not too large, the bond between the concrete and the rods would be sufficient to transmit that stress to the reinforcement and that it would be fairly well distributed between the concrete and the reinforcement, provided the rods are continuous. But, at the bottom of the column, we must have either a good square bearing of the rods upon adequate base plates, or they must be carried into the footing in some adequate way. The tests call into question the practice, more or less common,

Mr. Talbot. of having a portion of the rods rest merely on finished concrete as we go from floor to floor.

On the question of columns, we have now under way a set of tests on what we may call reinforced steel columns, to distinguish them from the ordinary reinforced concrete columns, where the steel is but a small percentage of the section of the column. In these, the structural section occupies 10 per cent. or more of the section of the column, the form being a riveted column, and the purpose of the test is to determine to what extent a steel column covered with concrete, will have advantage in strength and stiffness over a plain steel column. There are many who feel more reliance can be placed upon such forms of columns. Of course there are advantages in the way of rapid construction which come from the use of the steel section.

Web Reinforcement. Another set of tests is upon the effectiveness of web reinforcement in beams of such depth that the web stress or shear stresses are very important. In this we have made accurate measurement of stresses in a vertical direction and in a diagonal direction. There are those who say there is no vertical stress or tension or compression, and no diagonal compression. Our measurements, however, go to bear out the ordinary theory that we should expect from a combination of beam and truss action.

COMPARATIVE COST AND MAINTENANCE OF VARIOUS TYPES OF BUILDING CONSTRUCTION.

By J. P. H. Perry.*

The cost of an industrial building is not merely the sum of money paid to the architect and the builder for their services in designing and erecting the structure, but is actually this initial cost plus certain fixed annual charges which vary with the character of the building constructed. Ultimate cost is of extreme, though often lightly regarded, importance in determining the type of construction to be selected for a proposed factory or warehouse.

In many cases all of the following nine items have bearing on the ultimate cost of a building and in all instances some of these charges affect the owner's final expenditure on account of his building:

1. Interest.
2. Taxes.
3. Insurance on Building.
4. Insurance on Contents.
5. Depreciation.
6. Results of Vibration.
7. Effect of Increased Light and Sanitary Qualities.
8. Vermin Losses.
9. Fire Protection Value.

There are generally three classes of construction open to the choice of a prospective builder: Mill Construction, Steel Construction Fireproofed, and Reinforced Concrete.

Combinations of these three types are numerous and form exceptions to any cost comparisons which may be drawn. As for instance steel (or cast iron) columns and steel girders are used with heavy mill floors, or steel columns support reinforced concrete floors while the wall construction may be brick, concrete, terra cotta, or masonry. For the purpose of this paper, only the three distinct

*Manager, Contract Department, Turner Construction Company, New York, N. Y.

classes will be considered, their variations being so numerous as to prohibit detailed study. The relative cost of these three kinds of buildings may be stated to be thus:—

Mill Construction	100-130
Steel Fireproofed	154-140
Reinforced Concrete	143-128

In other words, reinforced concrete will cost from 10 to 20 per cent. less than structural steel (varying sometimes up to about equal cost with very light steel work) and from equal (or slightly less) cost to 10 per cent. more than first class mill construction and perhaps 30 to 40 per cent. more than the poor light mill or frame construction. This comparison of cost is based on data, given hereafter, gathered in reply to letters sent by the writer to 44 architects, engineers, and contractors, all of whom have designed or built numerous concrete buildings.

With some knowledge of what the initial cost of a proposed building will be attention should be directed to the items of annual expense already given. No contracts for a new structure should be awarded without careful study having been made by the investor of these yearly charges, for the reason that it is often possible to show an annual saving in cost on a building in favor of one type of construction despite the fact that another kind of building might cost quite a good deal less originally.

1. *Interest.* The amount of this charge varies directly with the initial cost of the building.

2. *Taxes.* The amount of this charge varies with the initial cost.

3. *Insurance on Building.* As a rule reinforced concrete buildings and steel buildings properly fireproofed will receive the same rate of insurance. Mill construction or frame construction buildings, as compared to reinforced concrete structures, run from equal rates to two or three times greater. Equal rates apply only where the rate on mill construction is so low that no lower rate can be given by the insurance company. It may be stated here that the mutual companies of New England are beginning to quote lower rates on reinforced concrete than they will on even the best of mill construction. Table I prepared by Mr. L. H. Kunhardt of the Boston Manufacturers' Mutual Fire Insurance Company, gives com-

parative rates on different classes of buildings. The data are taken from typical examples throughout the United States. The table is based upon absence of automatic sprinklers and other private fire protective appliances of the usual completely equipped building. They are not schedule rates but may be an approximation to actual costs under favorable conditions based on examples in various parts of the country.

An officer of another Mutual Fire Insurance gives the rates

TABLE I.—COMPARATIVE APPROXIMATELY YEARLY COST OF INSURANCE IN CENTS PER $100 OF FACTORY BUILDINGS.

Exposures none; area not large; good city department; no private fire apparatus except such as pails and standpipes.

	All Concrete.		Brick Mill Construction or Open Joists.		Wood Mill Construction or Open Joists.		Add for Brick or Wood Buildings in Small Towns and Cities without best of Water and Fire Departments.
	Bldg.	Contents.	Bldg.	Contents.	Bldg.	Contents.	
General storehouse	20	45	60	100	100	125	25
Wool storehouse	20	35	40	60	75	100	25
Office building	15	30	35	50	100	125	25
Cotton factory	40	100	100	200	200	300	50
Tannery	20	40	75	100	100	100	25
Shoe factory	25	80	75	100	150	200	50
Woolen mill	30	80	75	100	150	200	50
Machine shop	15	25	50	50	100	100	25
General mercantile	35	75	50	100	100	150	25

shown in Table II, taken from the report* of the Committee on Insurance of the National Association of Cement Users.

In connection with the matter of insurance on buildings, attention might be called to the results of the canvas made by the Committee on Insurance,† which showed that 26.6 per cent. of 255 owners of concrete industrial buildings carried *no insurance* on their building, and that a great many more would not carry insurance were it not for mortgages or loans requiring same.

4. *Insurance on Contents.* This expense varies almost directly with the character of the building. Of course some lines of industry are so constituted that the cost of insuring the contents of the buildings will remain disproportionately high, regardless of the character of the building.

Proceedings, Vol. VII, pp. 103-120.—ED.

†*Proceedings,* Vol. VI, p. 362.—ED.

5. *Depreciation.* Owing to changing conditions of American life and the comparative youth of the average business, exact data on depreciation are difficult to obtain. Structural steel buildings, properly fireproofed, are of so comparatively recent construction that depreciation figures are not available. It is the general feeling, however, that there is unquestionably some depreciation on structural steel buildings. Kidder's *Handbook* gives the annual depreciation of a mill building as varying from 1 to 1.5 per cent.; or, in other words, the life of a building is about 66 years. These figures include roofing, doors, windows, and other details. Matheson's *Depreciation on Factories* gives a table showing the result of very comprehensive study of English factory buildings for a period of many years and indicates that the value of a building costing $50,000, is at the end of 30 years $31,775, a depreciation of 36.6 per cent., or an annual depreciation of 1.2 per cent.

TABLE II.—AVERAGE COST OF INSURANCE ON REINFORCED CONCRETE BUILDINGS AND CONTENTS WITH SPRINKLER EQUIPMENT.

Agricultural Implement Factories	$0.06
Automobile Factories	.045
*Bakeries	.07
*Clothing Factories	.06
Lithographing and Printing Establishments	.055
Machine Shops	.04
*Metal-Working Plants, without Sprinklers	.06
Rubber Factories	.055
Shoe Factories	.05
Textile Mills	.05
Tobacco Factories	.06
Wood-working Plants	.07

Under this type of construction the insurance may be accepted not only at a lower rate, but under a considerable less valuation, probably resulting as whole in reducing the cost of the insurance from 40 to 50 per cent. over the ordinary type of factory and warehouse buildings.

*Risks not ordinarily open to competitive rates unless of superior construction.

The amount of depreciation on a concrete industrial building would be confined almost wholly to roofing, doors, and similar parts of the structure not of concrete. Though there are no records on which to base the estimate other than a knowledge of the permanency and the constantly increasing strength of the material as

well as the protection it affords the reinforcement, it may be conservatively stated that under the same conditions as given above for the mill building the annual charge for a sinking fund on a reinforced concrete factory or mill or warehouse would not exceed .25 per cent.

6. *Results of Vibration.* In the case of warehouses, foundries, and similarly used buildings the vibration element does not enter. With the ordinary industrial structure, however, one of the most important factors in a successfully operated plant is the keeping down of machinery repair and operation charges. Vibration in the building plays a considerable part in the amount of these charges. If machines placed in position will remain so and if shafting once lined up will not vary, the repairs to machinery, and more particularly the cost of operating the machinery, will be at a minimum. Vibration in a building causes "walking" of the machinery with consequent loss of alignment and increased wear on journal blocks and moving parts, and an increase in the amount of power necessary for operation.

Reinforced concrete has been chosen for proposed buildings in numerous instances in competition with structural steel and in a great many more cases in competition with mill construction, largely because of its vibration resistant qualities. While there are, it is true, some concrete buildings where vibration is noticeable, the conditions are such as to create vibration in any kind of a building and the amount existing in the concrete structure is probably less than would otherwise be obtained. There are many concrete buildings in which heavy machinery is operated on upper floors without any indication of their presence being given to the tenants in the lower stories.

To arrive at the annual value of the reduction or the elimination of vibration in a building, is somewhat difficult. The case of a paper goods manufacturer in Brooklyn, New York, the owner of both reinforced concrete and slow burning mill construction factories, bears on this subject. A saving of $5,000 per year in machinery repair and operation charges in favor of the concrete buildings compared with the mill buildings, is effected. This amounts to about 1 per cent. of the value of the building only, no allowance for contents.

In a concrete building occupied by a shoe machinery manufac-

turing concern, it was found that machines were being run at a much higher speed than ever possible before. The percentage of increased output was not doubled, but was very large indeed and the men operating the machines in their old building claimed the work was better in the new building, even with the increased speed.

In another large concrete building occupied by a book company of Cincinnati, Ohio, 15 or 16 heavy presses were operated on the second floor with perfectly satisfactory results. The owner stated that this would have been absolutely impossible in any other type of building.

The experience of another manufacturer, this time of buttons, occupying several concrete buildings, illustrates the effectiveness of concrete construction in reducing vibration. The officials of this company visited many concrete buildings before selecting this material for their present structures, and now state:

> We are running our machinery here at as high speed as we were ever able to in other buildings, and in general at higher speeds and with less waste time for adjustments and consequent repairs. I have no scientific data to furnish on this speed. . . . It is perfectly obvious that if one can get 5 or 10 per cent. more efficiency out of machinery, concrete buildings have a big advantage over others.

The Chief Engineer of this company writes as follows:—

> In the matter of our experience with vibration in our reinforced concrete building as compared with the vibration of the same equipment in a building of the ordinary mill construction would say:
>
> First, the type of vibration that concerns us the most is that due to very slight lack of balance in parts rotating at high speed, in distinction from the vibration that might be evident from machinery of a heavier type, due possibly to reciprocation as in heavy printing presses, or from shock as in punch press work and the like, or vibration of a machine tool due to chatter or like cause.
>
> In our experience previous to this building we had difficulty in anchoring our machinery so that this high frequency vibration would not in a short time loosen the fastenings as well as cause loss of adjustment in the machines themselves. We find this difficulty almost wholly overcome in our present concrete buildings, and further machines standing rather high from the floor which are subjected to more or less horizontal shock we were never able in a mill construction building, even by bolting through the floor, to prevent those machines from loosening. On our present concrete floors we have no record of any one of these machines requiring to be even tightened to the floor after being once installed.

As to building vibration, we never had experience with any building as free from jar, trembling, etc., as our present one, the results in all cases being a surprise to us in the elimination of these undesirable features.

In all plants where the amount of machinery installed was not as large as in these instances, the percentage saved based on the value of the building would be less. It may be assumed, however, for the purpose of comparison that in the case of the usual manufacturing plant, the saving by the elimination of vibration would amount at least to $1.50 per day, or $450.00 per year. When it is remembered that this includes power, oil, attendance, new bearings, parts, belts, and wear on the building structure, this estimate would seem conservative. The increased efficiency of the machines alone would in most cases equal or exceed this daily saving.

7. *The Effect of Increased Light and Sanitary Qualities.* It is well known that employees can work more efficiently in the open air, (assuming good weather conditions) than indoors for the simple reason that the ample light increases their efficiency. A garage owner recently stated he found that his men could repair an automobile when working out of doors in nearly one-half the time required when in his building. The effect upon the efficiency of employees of the amount of light obtainable is sometimes not appreciated by owners.

An increase of from 40 to 150 per cent. in the area of windows would certainly advantageously affect the working conditions in a building. Not only would the employees be able to see better, but also their spirits would be raised owing to more cheerful working conditions. The increase in the earning power of the working forces ought to be at least 1 per cent., in most cases it would be higher.

Welfare managers in many large manufacturing plants appreciate that the surroundings in which a man labors have a decided bearing on his output. Clean sanitary rooms with plenty of light, good ventilation and uniform temperature have a certain dollar and cents value. A saving of 1 cent per day per employee with 100 hands, represents $300.00 per year. One cent a day for a man earning $2.00 to $4.00 per day requires but a small increase in his effectiveness. Reinforced concrete buildings usually permit maximum light, owing to the method of construction. It is not unusual to find buildings of this material with 70 per cent. of their wall space

taken up by windows. It is possible to increase this light area to 80 per cent. when necessary. In mill construction as generally adopted, the window area does not run over 30 to 40 per cent. of the wall area. Cement being a lime product is essentially sanitary. The waterproof qualities of reinforced concrete render frequent washing of floors possible and make dampness or rotting impossible where wet processes are involved. The low conductivity value of concrete results in small variations in temperature within the building and the homogeneous construction largely prevents draughts. With these facts, not theories, as a basis, the assumption that the effectiveness of the employees will be increased at least 1 per cent. by using reinforced concrete in place of mill construction, would seem conservative.

8. *Vermin Losses.* Mice, rats, roaches, and other vermin are responsible for loss in most factories or warehouses. Raw materials or finished products are subject to inroads by these pests and although in well run plants the damage may be less than in others, it certainly has a monetary value. As an example of this, the experience of one paper goods manufacturer may be cited. With two mill buildings, and one reinforced concrete factory building for purposes of comparison he states that the last named structure saves him $4,000 per year in vermin loss alone. There is no place in a concrete building for rodents to live. An allowance of $100 loss annually would be too low for most businesses of sufficient magnitude to require a fireproof building.

9. *Protection from Fire.* The relative insurable value of a concrete building and a mill building has already been gone into. The protection from fire which a modern reinforced concrete industrial building affords its owner is probably the greatest asset which this form of construction possesses. Although a building and its contents may be fully covered by insurance, its owner cannot protect himself in event of fire against losses to his reputation owing to failure to complete orders or contracts; loss in the efficiency of his organization due to the acceptance by skilled operatives of other positions during a shutdown of the burnt out plant; and loss to his trade due to inability to accept new business until a new building can be procured. Beyond this protection against the ordinary fire concrete buildings offer a protection against conflagrations which cannot be estimated. In cases of great fires, a manufacturing

building, remaining practically unharmed, as has been the case with concrete buildings, would be of inestimable value. The certainty that a fire cannot destroy a building affects the confidence of employees and prevents panics and, incidentally, adds greatly to the peace of mind of the owner of the building. Many manufacturers occupying concrete buildings state that they can wake up in the night and hear fire engines go by without a fear that their plant is burning up. Concrete first attracted attention because of its absolute fireproofness.

TABLE III.—COMPARATIVE COST OF BUILDINGS OF CONCRETE AND MILL CONSTRUCTION.

Initial Cost of Building.	Mill Construction. $100,000.	Reinforced Concrete. $110,000.
Yearly charges:		
Interest at 6 per cent.	$6,000	$6,600
Taxes at 1 per cent.	1,000	1,100
Fire insurance:		
Building	at 75 cts. 750	at .25 cts. 275
Contents	at 100 cts. 1,000	at 80 cts. 800
Depreciation	at 1.25 per cent. 1,250	at .25 per cent. 225
Items charged against mill construction only:		
a. Loss due to vibration, assume	450	
b. Increased light, 1 per cent. increase in efficiency of labor. Assume labor equal to ½ value of contents or $50,000	500	
c. Vermin losses	100	
d. Protection against fire at .5 per cent. on value of 50 per cent. of building and contents	500	
Total	$11,550	$9,000
Annual saving of concrete over mill construction		$2,550

"Use and Occupancy" insurance policies can be obtained at a cost of about .5 per cent. on building and contents. They cover estimated profits, commissions, or similar legitimate sources of income, based upon the use and occupancy of the given building. In event of fire causing interruption to a business, the insurance company will pay the insured a certain amount for each day the building is out of commission. The value of a non-destructible building ought to be at least as much as this "Use and Occupancy" insurance. In fact it is worth a good deal more because a concrete building would not only make such insurance practically unnecessary, but would protect its owner against injury to reputation and organization. An assumption, therefore, that reinforced concrete

TABLE IV.—COMPARISON OF COSTS OF STEEL AND REINFORCED CONCRETE BUILDINGS.

Walls assumed the same.

Location of Building.	Size in Plan, ft.	Number of Stories.	Size of Panels, ft.	Use and Load in lbs. per sq. ft.	Character of Comparison.	Cost of Reinforced Concrete.	Cost of Fireproof Structural Steel.	Per cent. of Cost in Reinforced Concrete.	
								More than Steel.	Less than Steel.
Brooklyn, N. Y.	140 x 190	8, 2 basements	.....	Warehouse–200	Bid	$250,000	$280,000	.....	10.7
Boston, Mass (Sawtooth roof)	550 x 129	2, 1 basement	.....	Textile Mfg.	Bid and cost	278,289	286,445 Heavy mill floors	.....	2.8
	341 x 231	1, 1 basement	.....						
Cambridge, Mass.	60 x 320	5	.....	.....	Bid and cost	90,000	87,330 Wood floors	3.3	.....
Cincinnati, O.	.....	16, basement	.....	Office–70	Bid	20% lower for Structural Frame only	Entire building 4% higher than R. C.	.....	4.
Des Moines, Ia.	132 x 66	6	22 x 16.5	Factory–200	Estimate	60,650	69'750	.....	13.
Fairmount	50 x 100	2, basement	.....	Machine Factory–200	Bid	25,000	28,000	.....	10.7
Indianapolis	.....	5, basement	.....	Hospital	Bid	793,000	823,000	.....	3.6
Indianapolis	71 x 120	5, basement	16–8 x 20	Dept. Store–125	Bid	89,500	96,000	.....	6.8
Ohio	.....	12	.....	Mfg.–200	Bid	40,000 less than Steel	.....	.....	.....
Springfield, Ill.	283 x 105	8, basement	20 x 20	Loft Bldg.–150	Bid and Estimate	280,000	320,000	.....	12.5
St. Louis, Mo.	.....	10, basement	.....	Apartment–70	Bid	290,000	304,000	.....	4.6
St. Louis, Mo.	120 x 140	7, basement	.....	Hotel–70	Bid	171,000	184,000	.....	7.
St. Louis, Mo.	86 x 120	6, basement	16 x 18	Office–70	Bid	170,000	184,000	.....	7.6
Wichita, Kan.	75 x 140	10, basement	16 x 18	Office–70	Bid	282,000	303,000	.....	6.9

MEAN. Reinforced Concrete cost 6.4 per cent. less than Fireproofed Structural Steel.

TABLE V.— COMPARISON OF COSTS OF MILL CONSTRUCTED AND REINFORCED CONCRETE BUILDINGS.

Walls assumed the same.

Location of Building.	Size in Plan, ft.	Number of Stories.	Use and Load in lbs. per sq. ft.	Character of Comparison.	Cost of Reinforced Concrete.	Cost of Mill Construction.	Percentage of Cost in Reinforced Concrete.	
							More than Mill.	Less than Mill.
Boston, Mass.	120 x 155	9	Warehouse.	Bid	$196,000	$212,500	.....	6.3
Cincinnati, Ohio.	.....	.....	.....	Estimate	19,100	16,000	16.2	.....
Cincinnati, Ohio.	.....	.....	Press Building	Bid	4% more than Mill	4% less than Reinforced Concrete	4.	2.3
Cincinnati, Ohio.	60,000 sq. ft.	.....	Bakery–300	Bid	62,500	64,000	.....	.....
Detroit, Mich.	.....	3	Factory–300	Bid	28,500	28,200	1.5	.....
Fall River, Mass.	112 x 112	4	Factory	Bid	82,500	74,000	10.3	.....
Grand Rapids, Ia.	.....	.....	Factory	Bid	86,000	85,300	1.3	.....
Jersey City, N. J.	60 x 140	5	Factory–200	Bid	56,000	52,000	7.1	.....
Jersey City, N. J.	94 x 38	6, basement	Warehouse–200	Bid	43,000	39,000	9.3	.....
Manchester, N. H.	100 x 45	5	Factory	Estimate	72,000	52,000	27.7	.....
Nashua, Ohio	200 x 100	8	Storehouse	Bid	131,000	117,000	10.7	.....
New England	.....	.....	Shop	Estimate	69,539	65,868	5.2	.....
Pittsburgh, Pa.	100 x 120	4	Warehouse	Bid	63,636	61,489	3.3	.....
St. Paul, Minn.	165 x 235	7	Manufacturing–200	Bid	10,000 less than Mill	10,000 more than Reinforced Concrete	.....	.....

MEAN. Reinforced Concrete cost 6.7 per cent. more than Mill construction.

buildings will save .5 per cent. per year, owing to the protection which they give against fire, is permissible.

SUMMARY.

Table III sums up the various comparisons made between buildings of concrete and mill construction. Assume a building costing complete in mill construction $100,000, contents equal in value to the cost of the building and that it is used for general manufacturing purposes.

If the saving of $2,550 per year be capitalized at 6 per cent., it would represent a capital of $42,500. In other words, a concrete building, though 10 per cent. more in initial contract cost than a mill building of similar design, would save each year 2.5 per cent. over all fixed charges. Stated differently, an owner can afford to pay nearly 50 per cent. more for a concrete building than for a mill building and still save money on his annual output.

Careful consideration of Table III will show that the assumptions are conservative throughout. Several owners of large concrete factory buildings have commented on this table in much the same form as it appears and all have stated that in their judgment they could show a much greater saving in favor of the concrete building than appears from the table.

Tables IV and V give comparisons of the cost of actual buildings in structural steel, reinforced concrete, and mill construction.

The information contained in these tables is not authoritative, but is believed to be fairly accurate. The percentage of cost in favor or against reinforced concrete construction as shown in these tables, can not be applied as generalizations, as the number of cases cited is too small.

No comparison has been made herein between reinforced concrete and fireproofed steel construction such as has been attempted between concrete and mill construction for the reason that in most of the items considered there would be but little difference between a steel building and a concrete building. A modern reinforced concrete factory or warehouse is certainly the equal of a steel structure and can be constructed for considerably less money. It therefore does not seem necessary to enter into as close a study of the comparative working conditions in these two classes as it did between mill construction and concrete.

DISCUSSION.

Mr. Wm. H. Ham.—There is one item I want to call attention to which Mr. Perry has not emphasized very fully, and that is the economy in heating a concrete building. It may be interesting to note the result in a warehouse building of which the owner has given me records of temperatures. The building is one of a series in connection with a large plant, the only one of reinforced concrete, the others being generally of either old type wood joist construction or brick bearing wall buildings with steel roof trusses, generally of one story. Some, however, are covered with corrugated iron on the outside, used for blacksmith shop purposes, etc. With the same heating system the owner found a difference in temperature in the summer of at least 10 deg., and the same thing was true in the winter. The economy in the winter might or might not be a real saving, depending on whether there was an excess of steam for use in the plant and waste steam was used for heating. However, the lower temperatures in the summer in the concrete building could be figured as an economy, on account of the increased efficiency of labor with lower temperature. Mr. Ham.

One other item in the comparison between the mill concrete building is a little unfair as usually figured out, because the average mill building is intended for loads very much lighter than we are accustomed to in concrete construction, even with the additional factors of safety which our specifications for reinforced concrete design give. The fact of demand for larger load carrying capacity in the concrete building is one of the subjects to be studied very carefully. The average mill building for manufacturing purposes of New England is designed for a load of about 60 lbs. per sq. ft. The building law of Boston requires a building for light manufacturing to be designed for 125 lbs. per sq. ft.

The difference in cost in the City of Boston is about the same percentage as given by Mr. Perry. There is, however, one feature which should be brought into the case, and that is the possibility of longer spans in concrete than in mill construction. The loss of

Mr. Ham. space for manufacturing due to the added number of columns in a mill building is a point which should be figured into the economy of the two types of construction. I have just recently completed a plant in which figures were taken both ways. The percentage of increase in cost due to concrete was about as given by Mr. Perry. The number of columns in the building was reduced one-third. The light in the building was increased materially and the number of fire walls to satisfy the city and the insurance authorities was less.

Regarding vibration, I wish to call attention to what is probably one of the best mill constructed buildings in New England, where a very large number of looms are concentrated in several floors, one over the other, and the vibration due to the machinery was such that the walls were of necessity designed very much heavier than they would need to be for load carrying capacity. The result is the cutting off of light or raising a story. In this case the light was ample and the story had been raised to give the result. The vibration, however, was taken up by the walls instead of by the floors. The vibration is very perceptible on the floors, especially in the center of the building; and the cost of construction of this type of mill building is increased very much due to the weight of wall required to resist vibration. Often this is not taken care of and the building is allowed to vibrate.

Mr. Perry. Mr. J. P. H. Perry.—I would like to put on record a case which I heard of only recently. A first class mill construction building recently completed at a cost about $3,000 under the bids submitted by several reinforced concrete contractors on the same set of plans, has had $22,000 spent on it to stiffen it up in an endeavor to do away with the vibration. This amounts to very nearly 50 per cent. of the original contract cost.

Mr. Ballinger. Mr. W. F. Ballinger.—I fully agree in the main with Mr. Perry and it is the first time I have seen the comparison tabulated in the matter of cost, and the end, wherein he makes it about 23 per cent. cheaper in the long run to have a reinforced concrete building than a mill building, although costing 10 per cent. more in the first cost.

It might be fair to concrete construction, however, to say that if built in the country outside of the realm of building regulations the difference in cost may not be as much as 10 per cent. We have

had instances where we think it has cost no more to build a reinforced concrete building than a slow burning building. And again, in Philadelphia where the laws require more tower fire escapes and a smaller area of floor, with a division wall for every 15,000 sq. ft. if the building is of mill construction, whereas they allow 25,000 sq. ft. if it is fireproof construction, or omitting the fire wall and omitting the additional fire escapes where not required, sometimes just about offsets the difference in cost. Mr. Ballinger.

There is nothing further to be said as to the fact that there is a very great difference in the vibration between the mill building and the reinforced concrete building. But as to the difference in cost, we have one client who has written us that in one department, which is leather glazing, he has half the number of men, what they call fixers, to keep the machines in order since he has moved his machinery into the reinforced concrete building that he had when he was in the building of slow burning construction. He also says that it takes less power to run the machinery. He was buying his electric current in the old building and in the new building for a term, and with about the same number of machines running and about the same output found it took less power to run the machinery, probably due to the line of the shafting remaining true and the machinery staying in better order.

I do not quite agree with Mr. Ham's statement regarding the heating; that is, it does not concur with our experience. We find it takes more heat for a concrete building, especially in the top stories, where we have a large roof surface of reinforced concrete. We find that we have miscalculated our heating requirements in several instances and have had to put in additional heat to counteract the cooling effect of the concrete roofs; that is, more heat than we would have if the roof had been of slow burning construction. So I do not think it is safe to count on a saving of heat.

MR. HAM.—The radiation from the roofs is without doubt a very serious problem in the designing of any building, especially of a concrete building. The last method I have used in taking care of that is to apply on top of the roof a coat of cinder concrete, mixed 1 part cement, 10 parts cinders and no sand, this being 3 to 6 ins. thick, applied very loosely and leveled up with the usual float coat. I think with that or some similar insulator on the roof Mr. Ballinger's exception to the radiation would be eliminated. Mr. Ham.

Mr. Ham. Sometimes the crickets are formed in the cinder concrete, which is very porous, but gives a very desirable insulation. The warehouse and factory where records were taken, had a 6-in. flat slab roof without very much insulation in points. However, the crickets were formed of cinder concrete. So there was probably half the roof that was very well insulated by cinder concrete of a very lean mix. I think the leaner the mix the better in the insulation for a roof. It does not seem at first as if this concrete would set up solid enough to be of value. It takes sometimes 2 or 3 days before it is really in shape to apply the roofing. I have used the same for wall filling in several cases and have made insulating blocks for walls of the same mixture of cinder concrete and handled the block, large cakes, and placed them between outer and inner walls.

Mr. Wason. Mr. L. C. Wason.—Mr. Perry did not emphasize sharply the limit of loads between which concrete is cheaper or more expensive than mill construction. There is a factory built about ten years ago at Bayonne, N. J., the Pacific Coast Borax Works, where at the time it was built economy was the first consideration. The floors were designed to carry 800 lbs. per sq. ft. on 24-ft. spans. It was found that reinforced concrete was the only real cheap material they could use. It was cheaper than structural steel and much cheaper than wood.

In my experience the limit is about 200 pounds. That is, for much lighter loads mill construction is cheaper; for 200 pounds or more reinforced concrete is cheaper in first cost.

Secondly, in regard to lighting, Mr. Perry did not speak of artificial lighting. I had some investigations made in regard to that and found that it was usual for owners to whiten their ceilings and walls and that whitening held much better than it does on wood, so that you get a very good reflection and require less artificial light than you do with mill construction, which is somewhat of an advantage. Mr. Perry spoke only of natural lighting.

In an investigation which was carried on a year ago in regard to another purpose, namely, the relation between wooden and granolithic floors; several owners made the claim that it took a little more to heat the building up for the first time, but after it was once heated it took very much less heat to maintain a good even temperature than it did with mill construction, so

that the maintenance of heat after it was first established was cheaper than it was with mill construction. Mr. Wason.

Mr. Ham.—I would like to ask Mr. Wason if the load of 200 lbs. would not be variable, depending upon the span. It is my opinion that the long span beam crushes at a lower load than a short span. Mr. Ham.

Mr. Wason.—Why, yes, it is somewhat variable. Of course all conditions enter into that—locality, price of material as well as spans and loads. So when you say 200 lbs., that means in general the cost of spans somewhere around 20 ft. If you go into longer spans I think concrete is cheaper, and very short spans I think wood would be cheaper. Mr. Wason.

Mr. Ham.—That is, say with a span of 30 ft., the load would be very much less than 200 lbs. before the two buildings would be at the same price. Mr. Ham.

REGULATION RELATIVE TO DESIGN AND CONSTRUCTION IN REINFORCED CONCRETE.

Society of Hungarian Engineers and Architects, Budapest, November, 1909.

Section I.

In General. The term reinforced concrete structure here designates all such structures wherein the resisting power of iron and concrete is so combined as to jointly withstand the action of external forces.

These regulations shall not only apply to structures wholly of reinforced concrete, but also to such parts of other buildings in the construction of which reinforced concrete is used.

Section II.

1. Construction in reinforced concrete is based on a general and a detail design.

General Design. The requirements for the General Design are:

(*a*) Explanatory description with respect to the purpose of the structure, enumerating the several loads which served as the basis for the design.

(*b*) Drawing showing arrangements in a general way together with all data in reference to same; also diagram showing the placing of reinforcement, accompanied by detail statical computations.

Detail Design. The requirements for the Detail Design are:

(*c*) Detail drawings should be made of those sections only which are not sufficiently clearly shown in statical computations or in the diagram based on such computations.

(*d*) A report as to the building materials to be used, stating the quality of the ingredients of the concrete, the quality of reinforcement and giving name of place from which each has been originally obtained.

2. In addition to the signature of designer, the Detail Design must also be signed by the party contracting for the construction

in reinforced concrete, and he must do so before the commencement of the work at the latest.

3. The work of construction may commence upon the approval of the General Design; on those parts of the structure, however, of which—in accordance with Paragraph 1 (*c*)—a Detail Design is required, the work may commence only after such design has been submitted to the owner for inspection.

4. The designer is responsible for the correctness of the design of the structure, as well as for the statical computations in all detail; on the other hand, the party contracting for the actual construction shall be responsible for the quality of materials used, as well as for the proper execution of the work in accordance with the design. *Responsibility.*

Section III.

1. The weight of reinforced concrete shall be reckoned at 2,400 kg. per cu. m. (150 lbs. per cu. ft.); exception to this rule is permissible in such instances only where the weight has been definitely determined by proper authority. The weight of other parts of structure shall be taken according to their respective unit weight. **Loads.**

2. The effect of shocks caused by the nature of loading, as well as the effect produced by changes in magnitude of loading, must all be taken into account either by increasing the statical value of load or by reducing the allowable working strength.

In computations for railroad bridges a factor of 1.4 must be used for the moving load; for highway bridges a factor of 1.3 must be used. In the case of structures having machinery causing powerful shocks the factor should be 1.2 to 1.4.

3. When the structure, in addition to its own weight is also subjected to wind pressure, centrifugal force, shocks, change of temperature, or any other forces not mentioned here which nevertheless may cause internal stresses,—in these cases the effect of only those forces that may act simultaneously shall be considered in determining the maximum effect produced by such external forces; it should be observed that in these cases no factor is to be used in the computation for moving load.

4. The method for determining the reaction at supports, bending moment and shear, shall be the same as that used for corresponding structures made of one kind of material.

5. In the case of structures where the ends of the girders are firmly connected with their respective columns, and consequently those girders as well, which to a certain measure are joined with the columns,—with reference to the girder, the joint thus made may be generally ignored; that is to say, girders conforming to a straight line or those slightly deviating therefrom may be computed as free at both ends.

6. The effect of change of temperature must be carefully considered in connection with structural arrangements; whenever this cannot be neglected a change of $\pm$ 15 deg. C. ($\pm$ 27 deg. F.) must be allowed for in the computations, taking as a basis for the linear expansion of concrete the same as that of iron, 0.000012.

Bending Moment. Girder with 2 supports.

7. The maximum bending moment for a girder the ends of which are not connected with any degree of rigidity, must be determined the same as if both ends were free.

Should the arrangement and the proper design of structure require substantial connection at the ends, and if the designer fails to show the actual value of rigidity of the connection—then the positive bending moment for uniform loading at center of span must be computed by the formula $pl^2/10$, while the negative bending moment above points of support must be determined by $pl^2/15$, where l is the distance between supports and p is the load per unit length. For concentrated loading the maximum positive bending moment must be taken at 0.8 M'_{max} where M'_{max} represents the maximum bending moment. The maximum negative bending moment above point of support must be taken at not less than 0.4 M'_{max}.

Bending Moment. Girder with several supports.

8. In the case of continuous beams (whether actually such or in accordance with Paragraph 5 should be considered as such) the bending moments and stresses acting on the cross section must be computed by method applied to continuous beams when one kind of material is being used.

With a view to simplifying computations in less important cases, when negative bending moments at intermediate points of span either do not exist, or else are so insignificant that the same may be entirely ignored, and when the designer wishes to obtain approximate values for the bending moment at intermediate points of span,—then instead of applying the theory of the continuous beam the maximum bending moment (M_{max}) must be determined

by the following formulæ where l represents the greater of the two adjoining spans:

Uniform loading:

(*a*) At center of span ..... $+ M_{max} = pl^2/10$

(*b*) At cross section above intermediate supports for girders considered as having 4 supports ..... $- M_{max} = pl^2/8$

More than 4 supports ..... $- M_{max} = pl^2/10$

Concentrated loading:

(*c*) At center of supports ..... $+ M_{max} = 0.8\ M'_{max}$

Where M'_{max} equals the maximum bending moment of a girder having the same span.

(*d*) The maximum negative bending moment at intermediate columns must be given a value as great as the one holding good for the larger of the adjoining spans, that is, the value given should be equal to $+ M_{max}$.

9. If both ends of a girder having several supports are fixed and the designer fails to give accurate values for rigidity in relation thereto, then the approximate negative moment at points of support must be taken:

(*a*) For uniform loading, $pl^2/15$.

(*b*) For concentrated loading, one-half as great as the value of $+ M_{max}$, calculated for the end span in Paragraph 8 (*c*).

Span Length.

10. The distance between center line of supports must be reckoned as the span length. However, when the length of the portion of girder in bearing is greater than 1.5 times the minimum height of girder, the span length in such case must be taken equal to 1.5 times the height of girder, plus clear span.

Slabs.

11. The method given in previous paragraphs may be applied for approximating the statical values of slabs, with the exceptions:

(*a*) In computing for concentrated loading, the so-called "loaded square" must first be determined upon which—through the agency of pavement (covering)—the load may be regarded as uniformly distributed. (The method of determining the "square" for gravel covering is shown in Fig. 1; Fig. 2 shows the same for block pavement.)

(*b*) If the slab is supported by beams in one direction only,

the computed load must be determined by assuming the slab a girder supported on the edge, the working cross section of which (see Fig. 1)

$$s = a_1 + \frac{l}{3} + 2v_1$$

that is, it must be regarded as if the slab had absolutely no connec-

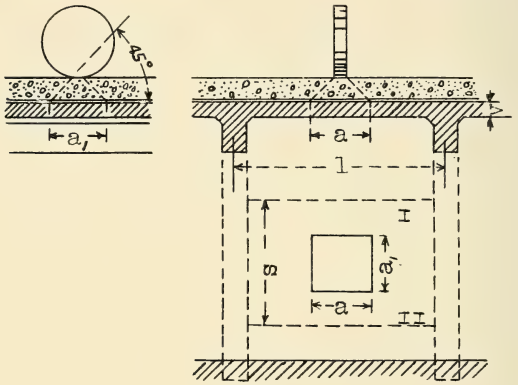

FIG. 1.

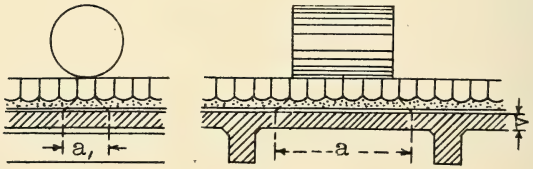

FIG. 2.

tion on the lines I and II. If the value obtained for s exceeds the value for l then the value of the latter must be taken instead of the value obtained for s. The value of the computed uniform loading (besides probable loading) at center of slab having length l and width s, must equal the concentrated load of the "loaded square."

The area of the cross section of reinforcement (as to unit width) in a direction parallel to the beams should be at least 20 per cent. of that required by computations in the direction l.

(c) If the slab is supported in two directions at right angles in such manner that the shorter span equals l, the longer l_1, and if $l_1 \geq 2l$, the slab must be regarded as if it were supported only by l, the shorter span; the span in the other direction being entirely ignored.

(d) If the slab is supported on all four sides and $l_1 < 2l$, then by method corresponding to the previous cases, that maximum bending moment must be taken into account which is obtained by first assuming the slab to rest on two parallel supports in one direction, and afterwards considering it in the other direction. Now then, if the moment at l, determined according to the above is M and that at l_1 equals M_1, the total value of the maximum bending moments (M' and M_1') which must be assigned to slabs supported on all four sides, is determined by the following formulas:

For short span $M' = \alpha M$

For long span $M_1' = \alpha_1 M_1$.

The value of the constants α and α_1 must be taken from Table I:

Table I.

l_1 / l	α	α_1
1.0	0.40	0.40
1.1	0.52	0.31
1.2	0.62	0.24
1.3	0.70	0.19
1.4	0.77	0.15
1.5	0.83	0.12
1.6	0.88	0.09
1.7	1.92	0.07
1.8	0.95	0.06
1.9	0.98	0.05
2.0	1.00	0.04

Intermediate values for the ratio l_1/l not given in Table I should be determined by proportion.

The area of cross section of reinforcement lengthwise (l_1) per unit length should be at least 20 per cent. of the amount required for the shorter span (l).

TABLE II. $D_{max} = \alpha P + \beta Q$.

l_2 / l_1	Girder with 3 supports. D_{max}	Girder with 4 symmetrical supports. D_{max}		Girder with 5 symmetrical supports. D_{max}			
	$\alpha = \beta$	α	β	α	β	α	β
0.2		0.317	1.063				
0.3		0.322	0.885				
0.4		0.328	0.755				
0.5	0.688	0.333	0.696	0.348	0.438	0.018	0.600
0.75	0.640	0.352	0.634	0.309	0.411	0.160	0.569
1.0	0.625	0.367	0.600	0.286	0.407	0.232	0.570
1.5	0.650	0.405	0.593	0.273	0.433	0.309	0.563
2.0	0.688	0.445	0.610	0.282	0.474	0.354	0.557
3.0	0.792	0.528	0.668	0.308	0.560	0.398	0.536

12. If the degree of rigidity of the connection of girders with columns can be disregarded in the computations with respect to the columns as well, then the axial compressive stress on the column may be computed in accordance with the theory of a continuous beam both ends being free. *Columns.*

In the case of uniform loading the value for the axial compressive stress on columns may be determined approximately as follows:

The maximum value of reaction of girders with two supports as well as those with three, four and five, and those symmetrically divided relative to center of girders is given by the formula:

$$D_{max} = aP + \beta Q$$

where P = the weight of girder uniformly loaded plus its permanent load; Q = the total of that probable uniform loading which produces the maximum reaction. The constants a and β —corresponding with the theory of a continuous beam—are to be taken from Table II. For value of conditions which come between those given in Table II intermediate values should be taken.

If the load Q (and perhaps to some extent P also) is not supported directly by the girder but is transferred by beams, etc.; then, in computing the stress on columns these must be regarded as the support of a simple beam, both ends free.

The maximum reaction on the end columns whether the girder has two or more supports, may be computed the same as if the girder extended across the end opening in the manner of a simple beam, having both ends free. This rule shall hold only for cases to which Table II may be applied.

Whenever a girder has more than five supports and the spans are equal, or else are very nearly symmetrical in relation to center of girder, then the maximum value of the reaction for intermediate supports may be computed by combining the two spans at each end of girder and consider it a girder with five supports. At such intermediate points where no support actually exists, the maximum reaction on a girder (having several supports) must be taken equal to the reaction of the central support of a girder assumed as having five supports. If the spans are unequal, then for each particular case we must determine (in order to find the maximum reaction at intermediate points) which particular ones of the four spans should be assumed as constituting the girder as one with four supports.

Section IV.

1. The ratio of moduli of elasticity of iron and concrete shall be expressed by **Computations.**

$$n = \frac{E_s}{E_c} = 15$$

Assumptions. 2. All computations for flexure as well as for all composite stress (flexure plus axial compression) shall be based on the following assumptions:

(*a*) The compressive stress on concrete is exerted in a direction parallel to a straight line.

(*b*) The stress produced by tension is carried wholly by the enclosed iron.

Working Stresses. 3. The maximum stress in any computation should never exceed the values in Table III.

TABLE III.

	Kg. per sq. cm.	Lbs. per sq. in.
Concrete.		
Compression		
(*a*) girders)		
(*b*) columns when loading is perfectly axial)	45	640.1
(*c*) columns, considering the deflection of girders)		
(*d*) columns, ignoring bending in computations	36	512
(*e*) columns, considering the simultaneously acting forces in computation as per Section III, 3	50	711.2
Shear	5	71.1
Bond between iron and concrete	6	85.3
Reinforcement.		
(*a*) Wrought iron		
Shear	950	13,512
Tension	1,200	17,068
(*b*) Welded iron.		
Shear	850	12,090
Tension	1,100	14,646

The maximum stress allowed in Table III for columns shall be valid only when value of m/v, relative length to thickness of columns, does not exceed 15; where m = length of column and v = dimension of shorter side. If the value of m/v is greater than 15 then, with a view to the danger of bending outward, the allowable stress for concrete must be reduced according to Table IV.

Tee Beams. 4. Whenever a slab is supported by means of beams in such manner that the slab and beam shall combine as one whole (tee-beam) and, if compression takes place in slab along the beam,—the width to be taken for the strained portion of slab should be: either $\frac{1}{3}$ of the span of slab or five times the height of beam (including the thickness of slab) or the distance between beams,—that is to say, whichever is the smallest of the three.

5. The area of reinforcement for columns should be generally considered at least 8 per cent. of the cross section of columns. In computing for axial compression then the entire area of cross section of the reinforcement must not be taken in account with a greater value than 4 per cent. of that of the columns. The distance between lateral support of reinforcement—so essential with respect to firmness of columns—must not be less than the shorter side of column, but under no consideration must such distance be less than 30 cm. (11.8 ins.). *Column Reinforcement.*

6. Should the compressive stress be eccentric then the bending moment originating therefrom must be duly considered. Table IV, Paragraph 3, shall here also serve (in accordance with m/v) to determine the allowable maximum stress. *Eccentric Stresses.*

Table IV.

m/v	15	20	25	30
The maximum compressive stress on concrete in columns..	σ_b	$0.8\sigma_b$	$0.6\sigma_b$	$0.5\sigma_b$

σ_b represents that value which as per Table III would be allowed if the value of m/v should be less than 15.

Intermediate values of m/v must be determined by proportion.

Section V.

1. For the supervision and direction of the actual work of construction the contractor must employ exclusively such individuals who are familiar with the methods of construction in all detail and who possess the practical experience required in laying out the work according to the plan and who also have the necessary training for the proper execution of the work on various parts of structure. In relation to all these points the contractor must furnish satisfactory evidence whenever requested by owner or authorities. **Supervision of Construction.**

Experienced workmen must be engaged on each individual part of work who, according to the extent of work, must be put in charge of a sufficient number of trained supervisors (foremen).

2. If the owner fails to have a representative permanently stationed on the work then the contractor must notify the owner in ample time in advance relative to the more important steps taken during the progress of work, for instance: the completion of the placing of reinforcement; time when placing of concrete is to begin; of the removal of forms and false work, etc.

SECTION VI.

Cement.

1. For reinforced concrete construction a good quality slow-setting Portland cement must be used.

2. The cement specifications issued by the Society of Hungarian Engineers and Architects shall be the standard with the difference that instead of 160 kg. (2,275 lbs. per sq. in.) the cement must have a compressive strength of at least 220 kg. (3,129 lbs. per sq. in.).

3. Upon request of owner or authorities the contractor must have tests of the cement delivered to work made at the State Experiment Station.

SECTION VII.

Aggregates.

1. Grains of material not exceeding 5 mm. (0.2 ins.) in size to be reckoned as sand; larger than this shall be counted as gravel.

2. The sand should be clean and be rather of mixed grain than uniformly coarse or—which is even much less desirable—of uniformly fine grain.

3. The gravel, whether obtained from river beds or by excavation, should also be clean and of such character as will render as few voids as possible, hence it should consist rather of mixed than of uniform size. Good quality of broken stone—natural or artificial—may also be used.

4. The largest dimension of gravel for concrete intended to encase the reinforcement shall also depend on the condition that it be not too large to pass between the reinforcement in place. Therefore, whenever conditions may require, the gravel of too large dimensions should be removed before mixing the concrete.

SECTION VIII.

Reinforcement.

1. The reinforcement used, as a rule, must be wrought iron.

2. The iron is of proper quality if:

(*a*) The pieces taken from the ends of the delivered rods (testing them on the work) give such satisfactory test results for flexure as are required by the rules for iron on State construction work.

(*b*) Wrought iron must have a tensile strength of at least 3,600 kg. (51,200 lbs. per sq. in.) and welded iron at least 3,300 kg.

per sq. cm. (46,900 lbs. per sq. in.), as to this strength the contractor must furnish satisfactory evidence to the owner whenever requested.

Section IX.

Concrete.

1. Each cubic meter of concrete placed must contain at least 300 kg. (18.7 lbs. per cu. ft.) of cement. The proportion of sand and gravel should be such as will render the smallest possible percentage of voids, which moreover must be determined by a comparative measurement system for voids; the volume of sand, however, should at least be $\frac{2}{3}$ of the volume of gravel.

2. The necessary proportion of sand and gravel must be taken by volume, while that of cement by weight; whether the mixing is done by machinery or hand, proper arrangements should be made in order that the weighing of cement may be carefully checked.

3. It is to advantage that the sand and gravel should be measured out separately; the use of the natural mixture of sand and gravel is also permissible provided the proportions of each correspond to what has been said in Paragraph 1, or has been made so by adding thereto the required amount of sand or gravel.

At short intervals during the progress of the work, the proper proportion of the two materials must be checked by screening and if found necessary the proper proportion of either to be added once more determined.

4. The quantity of water must be determined by the greater or less consistency required for the concrete, the difficulty encountered in the tamping process, the dampness of materials and atmosphere, the absorbing capacity of the forms, weather conditions as regards temperature, etc. Great precaution should be taken that the quantity of water thus fixed shall be strictly observed in the preparation of the concrete.

5. In the preparation of the concrete great care should be taken that the mixing of the aggregate, both before, during as well as after the addition of the water, should be most thorough, so that every single gravel in the mixture shall be surrounded with matrix. (As a rule the thorough mixing of concrete may be better attained by machine than by hand mixing.)

6. Until the methods for the preparation and examination of test cubes have been regulated, the quality of the concrete must be

judged with reference to above requisites; its preparation on the other hand must be judged with reference to the degree of faithfulness which the rules in these paragraphs have been adhered to.

Nevertheless it is desirable that concrete test cubes should be made of the proposed material before the commencemnt of work, in order that their toughness may be determined at the State Experiment Station. (Preliminary laboratory tests for concrete.) The tests are made on four cubes; each of these cubes should have a dimension of 20 cm. (about 8 ins.) and the concrete should be made with a sufficient amount of water to allow tamping. These tests are of particular importance in cases where the necessary experience regarding the proposed material is still lacking. The test is considered satisfactory if, disregarding the one rendering the poorest result, the three best ones give an average of 140 kg. per sq. cm. (1,991.2 lbs. per sq. in.) for the ultimate strength.

7. Whether preliminary tests have been made or not, the owner has a right to demand that the contractor, during the progress of the work, have made test cubes (of the concrete used) either in his or his representative's presence. The testing of these cubes (prepared on the work) must also be carried out at the State Experiment Station. However, the result of these tests may not come up to the rules laid down for the trial cubes in the regulations, and notwithstanding such failure it shall not be at any time in the future counted against the quality of the work of the contractor as long as the various materials used in the concrete were of proper quality and the construction was carried out in accordance with the regulations, and as long as those portions of structure—of which the test cubes were prepared—have given no reasons for anxiety either at the examination or at the trial loading preceding the final acceptance.

SECTION X.

Execution of Work.
Forms and Supports.

1. In order that the shapes and dimensions of the columns, beams, girders, etc., may strictly conform to the design, precautions should be taken that the supports of the forms are firm and have a solid footing; moreover, the forms should be so constructed as will permit the tamping of the concrete in thin layers. Further, the arrangement of the braces should be such that the removal of individual braces may be performed easily without shock, while

such braces as may be required for a longer period remain undisturbed.

2. All dirt, loose rust or grease should first be carefully removed from the reinforcement; the placing must be done accurately, in strict accordance with design and so as to maintain its proper position during the placing and tamping of concrete. Lengthening of iron by means of welding should be avoided, and is permissible at such points only where the stress is rather insignificant. In order to prevent possible slipping of the reinforcement within the concrete, the ends should be bent or else shaped in such other way as will answer the same purpose. *Reinforcement.*

3. The placing of the concrete must be in thin layers; particular care should be exercised that the entire surface of rods should be enveloped with a matrix of the concrete. If possible, the placing of concrete for beams and slabs should be carried through without interruption, and if, for some reason, this cannot be done, then before the work is resumed, the concrete already placed must first be cleaned and washed. In the selection of the point of interruption of work we must always be governed by the condition in the event of having failed to secure a perfect bond that the structure itself may not suffer shortage as to its load-bearing capacity. The placing of concrete may be interrupted, for instance, (*a*) on columns at any point; (*b*) on girders at the axis of the supporting columns, at the lower plane of the slabs supported by beams; (*c*) on slabs at the axis of the supporting beam. *Concrete.* *Joints.*

4. The newly placed concrete of the completed parts must be protected from the formation of minute cracks caused by rapid drying, and to this end, the concrete even under most favorable conditions must be kept continually moist for a period of at least one week.

5. Placing of concrete in freezing weather (– 4 deg. C., 24.8 deg. F.) shall be permissible only after proper arrangements have been made against the damaging effect of frost. The use of frozen concrete shall not be permitted under any consideration.

6. The braces placed for the purpose of preventing premature bending shall not be removed until the concrete has reached that required stage of firmness whereby it is capable of supporting its own weight. The time required for such firmness shall depend on the weather conditions prevailing during the progress of the work; *Removal of Forms.*

it further depends on the distance between supports, and the magnitude of the dead weight of the member.

As a matter of approximation, the following shall serve to show the shortest possible length of time under favorable conditions before the removal of braces:

(*a*) For such arch structures where the dead weight causes no greater internal stress than the probable loading and if greatest distance between supports does not exceed 7 m. (23 ft.), the length of time shall be two weeks after completion.

(*b*) Structures having a span greater than 7 m. (23 ft.) or whose dead weight exceeds that fixed in (*a*) must be supported for a longer period than two weeks: *i. e.*, girders, the dead weight of which is greater than twice the probable load and its span exceeds 10 m. (32.8 ft.) must be braced for at least four weeks. For intermediate cases, the permissible shortest length of time for bracing must be determined by taking a proper mean between the four and two weeks limit.

(*c*) If freezing weather sets in during the progress of hardening, then the length of time must be increased equal to the duration of the freezing weather.

Time of first Loading.

7. Reinforced concrete structures must not be loaded beyond the dead load (beams, braces, or posts supporting parts overhead), until at least four weeks after the completion and removal of the forms: and girders having a span over 7 m. (23 ft.) not until after six weeks. It is understood that even after that period, the loading must not exceed the amount of loading to which the respective part was intended, unless it be properly braced. Such structures for which trial loadings are stipulated must not be loaded in any shape or manner until after the trial loading.

For all damage to structure as well as for accidents due to premature removal of forms, the contractor shall be held responsible, and this liability shall not be diminished even if the time limit set in preceding paragraph has fully expired at the time of such removal of forms and braces; neither shall the degree of his responsibility be diminished by the circumstance that the owner's representative, per Section V, 2, has been duly notified and raised no objection to such removal of forms and braces.

8. With a view of obviating delay of the work, the structure—before the expiration of the time limit—can be loaded only if it be properly braced.

9. After removal of forms each individual part of structure must be examined as to:

(*a*) Cracks and other injury or damage as well as for any possible defect in construction.

(*b*) Whether the measurements of cross sections agree with design. Those individual parts which in the completed structure should become invisible or which by reason of removal of platforms should be difficult of approach—must be examined previous to the removal of the respective platform.

10. All defective and damaged parts must be removed and reconstructed in accordance with the method described, with the greatest possible care in each particular case and immediately executed; a detail description must be entered in the construction record (or proceedings) covering the actual carrying out of the reconstruction of the various parts of structure.

11. Care should be taken that openings necessary for conduits, fixtures, etc., shall not be made at points where they may be detrimental to strength.

Section XI.

Examination of Structure before Acceptance.

1. Previous to final acceptance and before any use is made of the reinforced concrete structure, the same must be examined by the owner or by the authorities; although portions of the platforms may still be in their places, notwithstanding the forms and false work must all be removed at the time of such examination.

2. The examination must consist of determining whether the measurements of cross sections agree in all details with those given in the design; whether cracks have appeared as well as to other possible damage or defect in construction. As for those parts of structure which are difficult of approach the result of the previous examination shall be observed. Sec. X, 9.

Test Loading.

3. In the examination of bridges and overhanging structures where the spans do not exceed 8 m. (26 ft.), as a rule no test loading shall be required, provided the design and execution of the work have been carried out under proper supervision and in accordance with these regulations. For all bridges and overhanging structures having spans greater than 8 m. (26 ft.), a test loading must be made before final acceptance and use.

The necessity of load tests for other kinds of structures shall

be determined for each particular case. The test must be made by all means in instances where the tests of the material used in the construction failed to come up to the requirements, or whenever either from observation or other data there is good reason for doubt concerning the proper execution of work on structure, or when through causes arising during the progress of work, the working strength of certain parts of structure may have been reduced.

4. A period of at least 6 weeks must elapse—after the placing of concrete—before the load test can be made.

5. If several spans of the structure are equal and if the method of construction on the various parts is similar, then the extent and the execution of the load tests shall be determined either by the owner or authorities. In general, the test loading should equal the probable maximum load for the respective part as given in the statical computations.

For sufficiently good reasons, the authorities or owner may demand the application of a test load in excess of this; however, the maximum load in such case cannot exceed the stress allowed in these regulations by more than 15 per cent.

In determining the magnitude of the test loading and in the execution of test, care should be taken that the pressure of supports on the soil shall not exceed the maximum amount allowed in computations.

6. The amount of change in form of structure due to test loading (flexure, displacement) must be measured not only immediately after the total load has been carried on or off the structure, but measurements should be taken at intervals later as well, until there should be no difference between two consecutive readings taken in one hour's duration.

7. During the progress of test loading, the structure must be carefully examined as to possible cracks.

8. The maximum amount of deflection must be determined as nearly as possible by means of computations as well.

It is generally desirable that during the test loading observations should be made of all such changes in form of structure which from a statical point of view may lead to additional information—this should be done even if the computation of the amount of deformation has as yet not been made.

The total deformation remaining must not be greater than one-third of the observed maximum deflection.

9. If during the test loading cracks appear to the extent of causing alarm or if the deformation exceeds the amount stated in the previous paragraphs, and further, if any of the material failed to come up to the requirements, and finally, if during test loading good reasons arise for anxiety, then the question of all additional tests and arrangements for safety shall be determined (with a view to the purpose of structure, the importance of the defects and other conditions, on the proposal of contractor after consultation with the designer or expert) by the owner.

If the load test has given entirely satisfactory results, except that the amount of deflection or strain observed is in excess of the amount computed, such variations shall not be considered objectionable if it be positively shown (for instance, by comparison with results of test loading for like structures, built for similar purpose) that such strain is unquestionably excessive.

Section XII.

Deviation from Rules.

(*a*) Stresses in excess of those fixed in these regulations shall be permitted in the computations only when a greater toughness of material than that prescribed in these regulations is assured.

(*b*) Whenever the statical value of any structure cannot be definitely determined by means of computation, such values may be deduced from proper experimental tests carried to the breaking point, provided both the construction and the loading were duly controlled by a State Experiment Testing Station. Such a structure shall be considered satisfactory if, at the time of test loading, the total load (own weight + fixed load + loading) placed thereon shall be equal to 3½ times the load (own weight + fixed load + probable loading) it is expected to bear.

Accepted by the Society of Hungarian Engineers and Architects at a regular meeting (512) held August 4, 1909, Budapest.

P. Balla,
First Secretary.

M. Kajlinger,
President.

RESOLUTIONS RELATIVE TO THE UNIFORM DESIGNATION OF THE BINDING MATERIALS USED IN THE PREPARATION OF CONCRETE

AND

OTHER RESOLUTIONS GOVERNING THE UNIFORM TRANSPORTATION AND EXAMINATION OF CEMENTS.

Society of Hungarian Engineers and Architects.

A. Uniform Designation of Materials Used for Concrete.

1. lime.

Lime is the product obtained from the calcination of limestones. When moistened with water it evolves great heat with extension of volume, resolving into fine powder. Upon the addition of more water it assumes with similar phenomena a paste-like consistency.

Lime is placed on the market either in lumps or, when hydrated, in powder form and also as slaked lime in pasty form, depending on local requirements.

Remarks.

The lime may be either rich or poor. The product obtained from crystalline or compact limestones containing not more than 8 per cent. of foreign substances and which when treated as above manifests a large increase in volume and is of smooth and greasy touch, may be considered rich lime.

If the limestones contain from 8-12 per cent. of impurities and if after calcination upon the application of water, they disintegrate to powder of uneven smoothness, such may be regarded as poor lime.

Poor lime should be kept in pits until used so that all solid particles may subsequently dissolve; this will result in the paste like quality being more uniform and of greater elasticity and besides the possibility of swelling and consequent cracking will be eliminated.

The placing of rich lime into pits is unnecessary because it may be purchased in powdered form and used at once.

If sufficient quantity of sand is mixed with either rich or poor lime the resulting mortar will harden within a certain length of time. The hardening process begins from without through the action of carbonic acid formed in such way that the water, held by hydration and mechanical action, is subsequently freed and evaporates. Lime mortar will not harden in damp places.

2. HYDRAULIC LIME.

The substance obtained from the calcination of argillaceous limestones or such as contain silicic acid is designated hydraulic lime. When moistened with water this material evolves slight heat and with no appreciable increase in volume either wholly or partially disintegrates to powder, which disintegration, although slowly, nevertheless takes place in time through the action of the air.

Hydraulic lime is placed on the market either in lumps or, when hydrated, in powder form, depending on local requirements.

Remarks.

Hydraulic lime is obtained from such argillaceous and siliceous limestones the hydraulic component of which is approximately 12 to 20 per cent. It is produced from the calcination of the materials to a point below incipient fusion. When slaked, the material either partially or wholly disintegrates to the fine powder, with slight increase in volume and moderate rise in temperature.

During the process of calcination the approximately 50 per cent. of oxide of lime contained in the material changes into powdered hydrate of lime. The raw material is combined with oxide of lime to an extent depending on its physical and chemical properties and on the success of calcination; portions of the raw material occasionally will not burn and must then be pulverized in mills together with the burnt material.

Such hydraulic lime which but partially disintegrates during calcination must not be sold in lump form.

Pulverized hydraulic lime has the touch of fresh soil and is of light yellow color with a shade of gray.

Hydraulic lime does not evolve heat when being prepared for mortar, its setting is always slow and its hardness increases both in air and in damp places.

Its specific gravity is between 2.8—2.9; when ignited the loss in weight is usually 8 per cent.

The mortar prepared from hydraulic lime attains hardness sooner than that prepared from lime, hence hydraulic lime may be used to

special advantage on dry and rapidly progressing construction work; in such construction where other lime is used the structure is apt in time to become damp and soaked with water.

3. ROMAN CEMENT.

Roman cement is the product obtained from the calcination to a degree below incipient fusion of argillaceous limestones (rich in clay). The clinkers will neither slake nor disintegrate upon the application of water, hence must be pulverized by mechanical means and the cement is usually placed on the market in that state.

Remarks.

Some argillaceous limestones possessing certain chemical and physical characteristics and containing from 20 to 50 per cent. hydrates, calcinated to a point below incipient fusion render Roman cement. The clinkers will neither slake or disintegrate through the action of air or water, hence must be pulverized by mechanical means.

The touch of the pulverized product is that of soil, the color varies from yellowish to reddish brown.

When being prepared for mortar the amount of heat evolved varies, depending on the quality of limestones used and the method of calcination. The time of setting is equally varying; with most Roman cements initial set develops within a few minutes and in course of time the hardness increases both in air and water.

The mortar prepared from a good quality of Roman cement must not change in volume, *i. e.*, it must neither swell nor disintegrate. The specific gravity of Roman cement varies and seldom reaches 3.0; the loss in weight due to ignition scarcely exceeds 5 per cent.

Roman cement may be used for both dry construction and water works, provided there is no danger of frost for a short time after its application. Roman cement as a rule is rapid setting and for this reason it may be used to particular advantage in such instances when a certain hardness of the mortar within a few minutes after its application is essential and at the same time a great degree of hardness is not necessary within such short space of time.

4. PORTLAND CEMENT.

Natural Portland cement is the pulverized product resulting from the calcination to incipient fusion of argillaceous limestones. The pulverized product resulting from the artificial mixture of marls of lime and clay renders artificial Portland cement.

Remarks.

Both the artificial mixture of marls of lime and clay or the natural argillaceous limestones will render Portland cement by means of calcination to incipient fusion.

The cement obtained from such raw materials which do not necessitate mechanical mixing, that is to say such as may be obtained from proper kinds of argillaceous limestones, are designated natural Portland cement; on the other hand such cements as are obtained by mechanical mixture, are designated as artificial Portland cement.

The raw material will not disintegrate either through the action of air nor water and must be pulverized by mechanical process. The pulverized product is decidedly sharp to the touch; the color is gray with a shade of green.

When being prepared for mortar Portland cement evolves heat but to a slight degree. As regards its setting qualities, it may be either rapid or slow; in time its hardness increases both in air or water; increase in volume does not take place to any extent whatever.

The specific gravity of good quality Portland cement is usually over 3.1; loss in weight due to ignition is less than 2 per cent.

In order that the technically important qualities of Portland cement may be regulated and investigated, the addition of 2 per cent. of foreign substance to the prepared cement shall be permissible without requiring a new designation (for such cement).

Portland cement must not contain more than 3 per cent. of magnesia.

Portland cement is serviceable both in dry and damp air as well as for all constructions under water; it is essentially serviceable, however, when considerable hardening, resistance to frost and friction is desired of the mortar in the shortest possible time.

5. MATERIALS PRODUCING HYDRAULIC QUALITIES.

As materials producing hydraulic qualities are designated all those natural and artificial materials which when mixed with lime as matrix will harden in both air and water.

Remarks.

The whole series of volcanic rocks of recent origin may be classified as materials producing hydraulic qualities: among these may be mentioned Trass prepared from suitable volcanic tufa (Trass rock), the Puzzuolan and Santorin earth, Paperino and other clay and silicate combinations. Such materials are used in the finely pulverized state.

6. PUZZUOLAN CEMENT.

Puzzuolan cement is obtained by means of intimately mixing powdered hydrated lime with materials producing hydraulic qualities.

Remarks.

By mixing (pulverized) materials producing hydraulic properties with (hydrated) powdered lime a certain hydraulic matrix is obtained which according to the grade of material used may—from a technical point of view—prove to be the most valuable of cements.

Such cements as belong to this class shall be always designated with respect to the material used as a base, viz.: slag cement, trass cement, etc.

The Puzzuolan cements when properly prepared lose their roughness and become soft and smooth to the touch; the color will correspond with that of the material producing hydraulic qualities used. During the preparation of mortar the Puzzuolan cements do not evolve heat and the rate of setting is generally slow. Well prepared Puzzuolan cements will not swell, but in consequence of rapid drying out are liable to crack.

Slag cement is the only Puzzuolan cement extensively used to-day. It is well adapted for all construction work—in dry air or under water —where an immediate great degree of hardness is not essential. Slag cement has decided advantages for concreting in water because of its slight tendency for sliminess. Some slag cements lose in strength when exposed to air because of cracking due to contraction and by reason of a reduction of the water of hydration. Slag cement may be generally regarded as unsuitable for such structures as are subject to friction.

The specific gravity of slag cement is variable to a considerable degree, depending on the material used as a base; it is usually less than 2.8. The loss in weight on ignition varies between 3 and 10 per cent.

7. MIXED CEMENT.

Mixed cements are prepared by adding certain materials to regular cements.

Remarks.

With a view of producing inexpensive binding materials of proper color, improve their fitness for sand, increase the degree of imperviousness and capacity to withstand frost, the mixing of foreign substances in suitable proportions to regular cement shall be permitted. However, when placed on the market such material must be plainly marked as "mixed cement" and the name of foreign substance contained therein designated.

B. Resolutions Governing the Uniform Transportation and Examination of Roman Cement.

Definition.—Roman cement is the product obtained from the calcination to a point below incipient fusion of argillaceous limestones (rich in clay). The clinkers will neither slake nor disintegrate upon the application of water, hence must be pulverized by mechanical means.

1. packing and weight.

Roman cement shall be placed on the market in barrels or bags and the price fixed per 100 kg. (220.5 lbs.) net weight.

Each barrel of cement shall contain 200 kg. (441 lbs.) cement net, and each bag 80 kg. (176 lbs.) net weight of cement.

A variation not exceeding 2 per cent. in the net weight shall not be objectionable.

The weight of the packing for the cement in barrels should not exceed 5 per cent. of the gross weight; for cement in bags the weight of packing must not exceed 1.5 per cent.

Each barrel or bag must bear the name of the manufacturer, should be marked "Roman Cement" and also with the net weight. In addition, the bags must be closed by means of lead seals, on one side of which the name of manufacturer must appear while on the other the words "Roman Cement."

Remarks.

For the purpose of simplifying the receipt of shipments of Roman cement and to avoid possible errors and misunderstanding it is always best to determine beforehand the weight to be allowed for each bag or barrel.

However, in exceptional cases the manufacturer is at liberty to make other arrangements with buyer relative to fixing weights.

2. rate of setting.

Roman cement may be either rapid or slow setting.

It is considered rapid setting if, when mixed with water (without sand) to normal consistency, it develops initial set in air in from 5 to 15 minutes after the addition of water. If a longer period than 15 minutes is required to develop initial set the cement is to be regarded as slow setting.

Remarks.

It is rather important to know the time necessary for the initial set to develop because the mortar or concrete prepared therefrom must be placed and incorporated in the work before initial set develops.

Apparatus.—The initial set as well as the hard set of Roman cement shall be determined by means of the standard needle, combined with the apparatus for measuring consistency of mixtures.

This apparatus has a metal face graduated in millimeters. There is a clamp holding a metal rod which can be set at any desired point vertically. A metal cup is attached at the upper end of the rod, while at the lower end is a brass cylinder, 1 cm. (0.39 in.) in diameter, the contrivance for determining consistency. The weight of the rod, cup and cylinder is 300 gr. (10.58 oz.).

The brass cylinder may be removed and exchanged for a standard needle. The needle is of metal with a plane end, the diameter of which is 1.13 mm. (0.044 in.), consequently the area is 1 sq. mm. (0.00155 sq. in.); before using the needle sufficient weights must be placed in the cup to make the total weight 300 gr., the weight of the needle being included.

The ring, constituting part of the apparatus, serves as the receptacle for the paste and is made either of hard rubber or metal, the diameter being 8 cm. (3.15 in.), the height 4 cm. (1.575 in.). When used it is placed on a glass plate which serves as the bottom of receptacle. When the cylinder rests on the glass plate, the pointer of the apparatus is at (0) zero, hence the distance between the end of the cylinder or needle and glass plate may be read on the graduated face of the apparatus.

The rate of setting of Roman cement is materially affected by the amount of water used, consequently in testing the setting characteristics the paste should be prepared in accordance with certain rules fixed for normal consistency.

Normal consistency of cement is determined by placing the ring filled with the paste together with the glass plate serving as the bottom of the ring, under the brass cylinder the end of which is gently adjusted to the surface of the paste, then allowing the cylinder under the pressure due to its own weight to pass into the cement paste. If the end of the cylinder remains stationary 6 mm. (0.236 in.) from the glass plate, that is to say if the pointer of apparatus is at the sixth line on the graduated face, then the cement paste is of normal consistency. With Roman cement such a consistency is usually attained by using from 40 to 60 per cent. of water by weight.

Methods.—The following methods should be observed in the examination of Roman cement:

Normal Consistency.—Four hundred grams (14.11 oz.) of the cement should be made into a thick paste with the aid of a spoon-shaped

trowel; if the cement is slow setting the mixing must not take longer than 3 minutes; if rapid setting the paste must be ready within 1 minute. This paste should be placed in the ring without tamping or shaking down, but should be carefully smoothed off. Then the ring holding the cement paste and resting on glass plate is placed, together with the glass plate, under the cylindrical rod and the test made.

If the desired result as to normal consistency has not been attained at the first trial then a new proportion of water must be tried until the proportion rendering normal consistency has been successfully determined. As soon as the required proportion of water rendering normal consistency has been established we may then proceed to ascertain the length of time required for initial set as well as for final set to develop in a cement of such consistency.

Time of Setting.—For this purpose the cylinder previously used shall be replaced by the standard needle.

In like manner as before the ring is filled with the paste of normal consistency and the needle is carefully lowered to the surface of the paste, repeating the operation at short intervals and at various points on top of the paste.

At the beginning the needle will penetrate the cement paste and touch the glass at the bottom; later, however, as the cement begins to set, the needle will stick fast in the paste.

When the needle fails to penetrate the entire thickness of paste, that period reckoned from the time of addition of water, must be regarded as necessary for *initial set* to develop.

As soon as the cement paste has hardened to such an extent that the needle leaves no appreciable mark on its surface, the time elapsed since the addition of water must be regarded as the period necessary for *final set* to develop.

Since the temperature of the air, as well as that of the water used, materially affect the rate of setting (high temperature accelerates while lower temperature retards), therefore, the tests must be made when the temperature of both air and water is at 15-18 deg. C. (59-65 deg. F.). If the tests are made under different circumstances then the temperature of both air and water must be especially noted.

As far as it is possible, the water used in the tests should be chemically and physically pure; above all it must be soft and free from sodium and iron.

Slow setting Roman cements must not evolve heat to any degree during setting; rapidly setting Roman cement may show appreciable rise in temperature.

In consequence of prolonged storage the rate of setting of Roman cement becomes slower and the cement loses its tendency for swelling; by storing it in weathertight storehouses the strength of its binding quality is improved.

3. CONSTANCY OF VOLUME.

Roman cement should retain its volume both in air as well as under water.

Remarks.

Some Roman cements increase in volume after the initial set has developed and in consequence the adhesion attained during the first stages of the setting is weakened, frequently resulting in cracks followed by disintegration.

This phenomena (swelling of cement) does not necessarily take place immediately after the hardening, but may appear after a shorter or longer period.

Roman cement cannot be regarded as having a constant volume unless it shall, without the addition of sand, permanently retain its original shape, both in air or under water.

Some Roman cement retains its shape under water and fails to do so in air, with others the case may be the reverse; hence, examination should be made of Roman cement in both directions with respect to this quality.

The examination concerning the constancy of volume of Roman cement shall consist of careful observations of test pats during a total period of 28 days, part of the time keeping them submerged in water and part of the time in air.

Method—Pats.—For this purpose four test pats are prepared, mixing the neat Roman cement on a glass plate with the water. The diameter of each pat shall be approximately 10 cm. (3.94 in.); thickness at center about 1 cm. (0.39 in.), tapering to a thin edge.

In order that the mixture may spread more freely on the glass plate, the percentage of water used for these tests should be somewhat greater than that allowed in tests for time of set.

That the formation of cracks on test pats—due to rapid drying out—may be avoided, they should be well protected from air currents and the heat of the sun. This is easily accomplished by keeping the test pats in a moist closet where they should remain at least 24 hours and by all means until final set has developed. Thereafter two of the test pats on glass plates should be kept in air in a convenient place, well protected from the heat of the sun and drafts of air; the other two are placed in water; all four test pats are to remain undisturbed for 27 days.

Should any of the test pats show signs of distortion or should cracks appear (running from edges towards center) then the tendency of the cement for swelling is unquestionable. On the other hand, if the test pats show no change, respecting shape or form, the same shall prove the constancy of volume, both in air and water.

4. FINENESS.

Two sieves shall serve to test the fineness of cement; one having wires 0.10 mm. (0.00394 in.) in thickness and containing 900 meshes per sq. cm. (about 76 meshes per lin. in), the other made of wires 0.07 mm. (0.00276 in.) thick and with 2,500 meshes per sq. cm. (127 meshes per lin. in.).

The fineness of Roman cement should be such that, when it is passed through the sieve having 900 meshes per sq. cm., the residue shall not exceed 18 per cent., and the portion of cement passing the above sieve passed through the sieve having 2,500 meshes per sq. cm. shall not leave a residue exceeding 36 per cent. Hence, the total material tested and passed through both sieves shall not leave a residue exceeding 54 per cent.

Remarks.

As Roman cement is used in combination with sand and broken stone and since the strength of binding, toughness and imperviousness of the concrete increases with the fineness of grinding of cement (the coarse grains merely take the part of sand) we should insist on having cement of a finely pulverized quality, as to which examination should be made by means of the described combination sievings.

One hundred grams (3.52 oz.) of Roman cement should be taken for the test sievings; the sieving shall continue until the dropping of cement ceases completely, which can be determined by repeated sieving over a white sheet of paper.

It would be a mistake, however, to judge the good quality of cement solely by the fineness, since a great degree of fineness in the case of soft and poor quality of cement is of greater frequency than for those of the better grade; moreover, should the pulverization of the latter be coarser, yet the strength of binding is often greater than that of the finely ground poor grade cements.

5. STRENGTH.

The strength of binding of Roman cement is to be determined on a mortar prepared of sand and Roman cement.

The standard mixture is three parts of standard sand and one part of Roman cement, taken by weight.

The examination shall consist of tests for both tension and compression. The test pieces in each respective class shall be of the same shape and dimension, and the test must be made by means of well constructed apparatus.

Tests for compression shall be made after 28 days of hardening. The uniform good quality of the material shall be controlled by means of tests for tension after 7 and 28 days of hardening.

The tests for tension must be made on test pieces, Fig. 1, the area of the strained plane of which must be 5 sq. cm. (0.775 sq. in.) [2.25 cm. (0.886 in.) long and 2.22 cm. (0.873 in.) wide]; tests for compression must be made on test pieces having an area of 50 sq. cm. (7.75 sq. in.) [length of each side 7.07 cm. (2.78 in.)]. After the test pieces are prepared, they shall be kept in air for 24 hours and thereafter in water until tested.

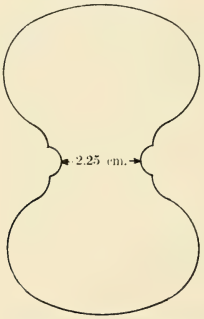

FIG. 1.—FORM OF TENSION TEST PIECE.

Remarks.

Inasmuch as Roman cement is almost exclusively used in combination with sand, its binding quality must also be determined in a similar mixture.

Three parts of sand and one part of cement renders a convenient proportion whereby the quality of Roman cement may be satisfactorily determined.

Experience shows that the strength of concrete is materially affected by the chemical and physical properties of the sand used, therefore, in order to obtain good comparative results, the sand used in the preparation of each and every test piece shall be of the same grain, weight and quality.

Such standard sand is obtained by first thoroughly washing a pure quality of silicious sand and after drying, passing it through a sieve having wires 0.40 mm. (0.0158 in.) diameter and 64 meshes per

sq. cm. (20 meshes per lin. in.), thereby removing the coarse pieces; then passing the sand through a sieve having wires 0.30 mm. (0.0188 in.) diameter and 144 meshes per sq. cm. (30 meshes per lin. in.), removing the finer grains.

The residue of this sieving will render the standard sand.

The tests for compression are standard tests because in practice the concrete is mostly subjected to compressive stresses. The ratio of compressive strength to tensile strength varies with different cements and definite deductions as to compressive strength cannot be made from the tensile strength of a cement. Tests for compression must be made on test pieces, after 28 days of hardening, because the quality of some cement is not sufficiently evident until after that period.

It is advisable to extend the time limit whenever possible, for it frequently occurs that while some cement may attain but a limited degree of strength at the beginning, later it will not only equal but exceed the toughness of other cement.

The uniformity of quality of shipments of Roman cement may be determined by means of tension tests; 7 and 28 day test pieces, respectively, should be used for these determinations.

The 7 day test pieces are used in order that the relative value thereof may be compared with the results obtained for compression on the 28 day test pieces, and by using such relative values as a basis, such other data may be obtained as will enable a deduction of the standard value for toughness. The 28 day test pieces used for tension give information in regard to the rate of setting.

In order to obtain a better comparison of Roman cement with the other varieties of cement, it is advisable to test the tensile strength of the neat cement because some cement when in a pure state will render a better account of its peculiarities.

In connection with tests for toughness, the weight per liter of the loosely sieved sand must also be noted in each case. The sand must be passed into a cylindrical vessel 10 cm. (3.94 in.) high, the volume being one (1) liter; the sieve used has wires 0.40 mm. (0.0158 in.) thick and 64 meshes per sq. cm. (20 meshes per lin. in.), and is held 15 cm. (6 in.) above the top of the vessel during the operation of sieving.

The sieving should continue until the base of the cone formed by the sand on top of vessel extends out to the rim and should then be carefully smoothed off with the aid of a straight smoothing iron.

During the entire process, shaking the liter measure should be carefully avoided.

6. TENSILE AND COMPRESSIVE STRENGTH.

The standard mixture for Roman cement (1 part cement and 3 parts standard sand, both by weight, 28 days of hardening, first

24 hours in air, 27 days in water) must have a tensile strength of 10 kg. per sq. cm. (142.2 lbs. per sq. in.), and 80 kg. per sq. cm. (1,138 lbs. per sq. in.) in compression.

Of six tests, the average of the four best ones shall be taken as the result.

Remarks.

1. PREPARATION OF TEST PIECES.

(*a*) *General Remarks.*

Compression test pieces must be made by machine, while tension test pieces may be prepared either by machine or hand. In disputed cases, the results of the machine made test pieces shall be the criterion.

For testing the toughness of cement, six test pieces must be made for each period of hardening.

The sand and Roman cement must be thoroughly mixed when dry. The sand is first weighed and deposited in vessel and the cement placed on top. These are intimately mixed with the aid of the trowel and then the water is poured evenly over the entire surface and the mortar mixed from 1 to 3 minutes according to time limit, but by all means until intimately mixed. The mortar is then placed into well cleaned and moistened forms resting on metal plates; the subsequent depositing of remnants of mortar into the molds must not be permitted.

The test pieces must by all means be ready before the initial set develops, therefore, with rapid setting cements in particular, special care must be exercised in the preparation of the test pieces.

The locks on the molds for tension test pieces should be of such form as will thoroughly resist the pressure to which the test pieces are subjected when being prepared; should the locks become loose, the area of cross-section to which the stress is applied, becomes larger and consequently the result of the tests would be incorrect; therefore the method of connecting the two parts of molds by means of springs should be rejected.

(*b*) *Preparation of Test Pieces by Machine.*

To obtain correct comparative results between compression and tension tests it is necessary to prepare the mortar of the same consistency and compactness. This result is obtained by adding the same quantity of water and mixing the dry materials in identically the same manner in each case.

The best means for determining the proportion of water is by taking mixed dry material 740 g. (26 oz.) in weight and arbitrarily selecting the quantity of water, thoroughly mixing same, being careful that for quick setting cement the mixing should be finished within 1

minute, for slow setting cements, within 3 minutes; immediately after mixing the mortar is placed into the mold, compacting it by applying 150 blows of the 3 kg. (6.6 lb.) ram dropped from a height of 50 cm. (19.7 in.).

Should a slight watery slime appear on the surface of the test piece after it has been subjected to 120-130 blows of ram, the same indicates that the quantity of water used was a correct proportion; should this not be the case then the experiment must be repeated until the above result is attained.

The amount of water thus determined and expressed in per cent. will render the mortar of standard consistency and all test pieces shall be prepared accordingly.

Each test piece must be prepared separately; the quantity of dry materials for compression should be 740 g. (26 oz.), that for tension 160 g. (5.65 oz.), to be mixed with the proper percentage of water as described above.

The mortar is deposited in the molds. The test pieces intended for tension shall be rammed with 120 blows of the 2 kg. (4.4 lb.) ram dropped from a height of 25 cm. (9.85 in.); those intended for compression with 150 blows of the 3 kg. (6.6 lb.) ram dropped from a height of 50 cm. (19.7 in.). Immediately after the last blow of the ram the plug and upper portion of the mold is removed, the excess of material cut away and smoothed off and when hard set develops the test piece taken out.

The appliance for compacting the test pieces must have a solid footing; it should preferably rest on masonry.

(*c*) *Preparation of Test Pieces by Hand.*

Each test piece should be prepared separately.

For each test piece 40 g. (1.41 oz.) of Roman cement and 120 g. (4.2 oz.) of sand are thoroughly mixed dry in a vessel and a quantity of water, equal to about 10 per cent. of the dry materials by weight, is added so as to give the mixture the consistency of putty.

With the aid of a trowel 35 cm. (13.79 in.) long and weighing 250 g. (8.8 oz.) the mixture is placed into the mold resting on a sheet of metal or glass plate in such manner that the mixture overtops the vessel; it is then tamped into the mold, using for this purpose the flat surface of the trowel, which should be 3 cm. (1.18 in.) wide and 5 cm. (1.97 in.) long.

The tamping at the beginning should be gently around the sides and then with increased force until water appears on the surface of the mixture. The ramming must by all means be continued until water is forced to the surface of the mixture, which requires about one minute.

The material extending above the mold is then cut away and the upper surface of mortar smoothed off. After the development of hard set the molds are carefully removed.

As a comparison of the quality of hand made test pieces, each series should be weighed both on removal from the water as well as on removal of molds; the average of the two weighings should be noted in connection with the results of the experiments. The weights of test pieces in one series should be about the same and should not be less than 150 g. (5.3 oz.). Increase in weight in 28 days due to absorption of water should not be more than 3 per cent. Test pieces lighter in weight or which due to absorption of water reveal a greater increase in weight than stated should be rejected.

(d) Preparation of Tension Test Pieces of Neat Cement.

The mold should be slightly oiled on the inner surface and placed on a sheet of metal or glass plate. One hundred and sixty grams (5.664 oz.) of Roman cement is mixed with water. If the test pieces are to be made by machine, the proportion of water must be determined by the experimental method described in paragraph (*b*), if by hand the proportion of water shall be taken as described in paragraph (*c*). The cement should be mixed as long as the initial set (approx. 5 minutes) will permit. The mold is then filled well over the top with the mixture. Then proceed as prescribed for test piece made of standard mixtures.

The molds should be removed only after the paste has sufficiently hardened.

Quick setting or very finely pulverized Roman cement will necessitate the use of a larger quantity of water in order to allow ramming to proper compactness; when the results of these test pieces are reported the amount of water used in preparing them should be stated.

2. STORAGE OF TEST PIECES.

During the first 24 hours the test pieces should be kept in air; to prevent uneven drying out they should be placed in a damp closet and after period stated should be deposited and left in water until the time of testing.

The water in which the test pieces are deposited should be renewed weekly during the first four weeks; there should be sufficient water to keep the test pieces continually submerged. If the test pieces are stored for a long period a fresh supply of water should replace the portion evaporated.

3. MAKING THE TEST.

The tests must be made immediately upon taking the test pieces out of the water. For tension and compression 6 tests shall be made, each taken from the respective series of preparation. The average of the four best ones in each case shall be taken as the result of the test.

Since the rate of application of stress in tests for tension affects the result, the gradually applied stress should be at the rate of 100 g. per sec. (0.22 lbs. per sec.).

When placing the test pieces into the clip care must be taken that the force applied should act in a direction perpendicular to the plane of the strained area, *i. e.*, it should be well centered and not subjected to cross strains; the best means to attain this result is by having the contact points of the clips slightly curved and not straight, so that the test pieces have only two points of contact.

With a view to uniformity in results of compression tests, the pressure should always be applied on the sides perpendicular to the direction of ramming and never on the rammed surface and side opposite.

The methods and results of the Cement Testing Station at the Royal Joseph University of Budapest shall be standard in all cases of dispute.

C. RESOLUTIONS GOVERNING THE UNIFORM TRANSPORTATION AND EXAMINATION OF PORTLAND CEMENT.

Definition.—Natural Portland cement is the pulverized product resulting from the calcination to incipient fusion of argillaceous limestones. Artificial Portland cement is the pulverized product resulting from the artificial mixture of marls of lime and clay.

In order that the technically important qualities of Portland cement may be regulated and investigated, the addition of 2 per cent. of foreign substance to the prepared cement shall be permissible without necessitating a new designation.

Portland cement must not contain more than 3 per cent. of magnesia.

1. PACKING AND WEIGHT.

Portland cement shall be placed on the market in barrels or bags and the price fixed per 100 kg. (220.5 lbs.).

Each barrel of cement shall contain 180 kg. (397 lbs.) cement net and each bag 60 kg. (132 lbs.) net weight.

A variation not exceeding 2 per cent. in the net weight shall not be objectionable.

The weight of the packing for the cement in barrels should not exceed 5 per cent. of the gross weight; for cement in bags the weight of packing must not exceed 1.5 per cent.

Each barrel or bag must bear the name of the manufacturer, should be marked "Portland Cement" and also with the net weight. In addition, the bags must be closed by means of lead seals, on one side of which the name of manufacturer must appear while on the other side the words "Portland Cement."

Remarks.

For the purpose of simplifying the receipt of shipments of Portland cement and to avoid possible errors and misunderstanding, it is best to determine beforehand the weight to be allowed for each bag or barrel.

However, in exceptional cases, the manufacturer is at liberty to make other arrangements with buyer relative to fixing weights.

2. RATE OF SETTING.

Portland cement may be either rapid or slow setting.

It is considered rapid setting if when mixed with water (without sand) to normal consistency, it develops initial set in air in from 10 to 30 minutes after the addition of water. If it requires a longer period than 30 minutes for initial set to develop the cement is to be regarded as slow setting.

Remarks.

It is rather important to know the time necessary for the initial set to develop because the mortar or concrete prepared therefrom must be placed and incorporated in the work before the initial set develops.

Apparatus.—The initial set as well as the hard set of Portland cement shall be determined by means of the standard needle, combined with the apparatus for measuring consistency of mixtures.

This apparatus has a metal face graduated in millimeters. There is a clamp holding a metal rod which can be set at any desired point vertically. A metal cup is attached at the upper end of the rod, while at the lower end is a brass cylinder, 1 cm. (0.39 in.) in diameter, the contrivance for determining consistency. The weight of the rod, cup and cylinder is 300 gr. (10.58 oz.).

The brass cylinder may be removed and exchanged for a standard needle. The needle is of metal with a plane end, the diameter of which is 1.13 mm. (0.044 in.), consequently the area is 1 sq. mm. (0.00155 sq. in.); before using the needle, sufficient weights must be placed in the cup to equal 300 gr., the weight of needle being included.

The ring, constituting part of the apparatus, serves as the receptacle for the paste and is made either of hard rubber or metal, the diameter being 8 cm. (3.15 in.); height 4 cm. (15.75 in.). When used it is placed on a glass plate which serves as the bottom of receptacle. When the cylinder is lowered to the glass plate, the pointer of the apparatus is at (0) zero, hence the distance between the end of the cylinder or needle and the glass plate may be read on the graduated face of the apparatus.

The rate of setting of Portland cement is materially affected by the amount of water used, consequently in testing the setting characteristics the paste should be prepared in accordance with certain rules fixed for normal consistency.

Normal consistency of cement is determined by placing the ring filled with the paste together with the glass plate serving as the bottom of the ring, under the brass cylinder the end of which is gently adjusted to the surface of the paste, then allowing the cylinder under the pressure due to its own weight to pass into the cement paste. If the end of

the cylinder remains stationary 6 mm. (0.236 in.) from the glass plate, that is to say, if the pointer of apparatus is at the sixth line on the graduated face, then the cement paste is of normal consistency. With Portland cement such a consistency is usually attained by using from 27 to 30 per cent. of water by weight.

Methods.—The following methods should be observed in the examination of Portland cement:

Normal Consistency.—Four hundred grams (14.11 oz.) of Portland cement should be made into a thick paste with the aid of a spoon shaped trowel; if the cement is slow setting the mixing must not take longer than 3 minutes; if rapid setting the paste must be ready within 1 minute. This paste should be placed in the ring without tamping or shaking down, but should be carefully smoothed off. Then the ring holding the cement paste and resting on glass plate is placed, together with the glass plate, under the cylindrical rod and the test made.

If the desired result as to normal consistency has not been attained at the first trial then a new proportion of water must be tried until the proportion rendering normal consistency has been successfully determined. As soon as the required proportion of water rendering normal consistency has been established, we may then proceed to ascertain the length of time required for initial set as well as for final set to develop in a cement of such consistency.

Time of Setting.—For this purpose the cylinder previously used shall be replaced by the standard needle.

In like manner as before the ring is filled with the paste of normal consistency and the needle is carefully lowered to the surface of paste repeating the operation at short intervals and at various points on top of paste.

At the beginning the needle will penetrate the cement paste and touch the glass at the bottom; later, however, as the cement begins to set, the needle will stick fast in the paste.

When the needle fails to penetrate the entire thickness of paste, that period reckoned from the time of addition of water, must be regarded as necessary for *initial set* to develop.

As soon as the cement paste has hardened to such an extent that the needle leaves no appreciable mark on its surface, the time elapsed since the addition of water must be regarded as the period necessary for the *final set* to develop.

Since the temperature of the air as well as that of the water used materially affect the rate of setting (higher temperature accelerates while lower temperature retards), therefore, the tests must be made when the temperature of both air and water is at 15-18 deg. C. (59-65 deg. F.). If the tests are made under different circumstances then the temperature of both air and water must be especially noted.

As far as it is possible, the water used in the tests should be chemically and physically pure; above all it must be soft and free from sodium and iron.

Slow setting Portland cement must not evolve heat to any degree during setting; rapidly setting Portland cement may show appreciable rise in temperature during setting.

In consequence of prolonged storage the rate of setting of Portland cement becomes slower and the cement loses its tendency for swelling; by storing it in weathertight storehouses the strength of its binding quality is improved.

3. CONSTANCY OF VOLUME.

Portland cement should retain its volume both in air as well as under water.

Remarks.

Some Portland cement increases in volume after the initial set has developed and in consequence the adhesion formed during the first stages of the setting is weakened, frequently resulting in cracks followed by disintegration.

This phenomena (swelling of cement) does not necessarily take place immediately after the hardening, but may appear after a shorter or longer period.

Portland cement cannot be regarded as having a constant volume unless it shall, without the addition of sand, permanently retain its original shape, both in air or under water.

Some Portland cement retains its shape under water and fails to do so in air, with others the case may be the reverse; hence, examinations should be made of Portland cement in both directions with respect to this quality.

The examination concerning the constancy of volume of Portland cement shall consist of careful observations of test pats during a total period of 28 days, part of the time keeping them submerged in water and part of the time in air.

Method—Pats.—For this purpose four test pats are prepared, mixing the neat Portland cement on a glass plate with the water. The diameter of each pat shall be approximately 10 cm. (3.94 in.); thickness at center about 1 cm. (0.39 in.), tapering to a thin edge.

In order that the mixture may spread more freely on the glass plate, the percentage of water used for these tests should be somewhat greater than that allowed in setting tests.

That the formation of cracks on test pats—due to rapid drying out—may be avoided, they should be well protected from air currents and the heat of the sun. This is easily accomplished by keeping the test pats in a moist closet where they should remain at least for 24 hours and by all means until final set has developed. Thereafter, two of the test pats on glass plates should be kept in air in a convenient place,

well protected from the heat of the sun and drafts of air; the other two are placed in water; all four test pats are to remain undisturbed for 27 days.

Should any of the test pats show signs of distortion or should cracks appear (running from edges toward center) then the tendency of the cement for swelling is unquestionable. On the other hand, if the test pieces show no change respecting shape or form, the same shall prove the constancy of volume both in air and water.

Boiling Test.—Should the time for determining the constancy of volume of Portland cement be limited then the boiling test is applied.

At least two test balls, each 4 or 5 cm. (1.5 or 2 in.) in diameter, are prepared of neat cement, the paste being of normal consistency. When the test balls are ready they should be deposited in a moist closet for at least 24 hours, but by all means until the cement sets.

The balls are then placed on a screen in a brass vessel of 20 cm. (7.87 in.) diameter and 24 cm. (9.45 in.) high, filling the vessel with water so as to keep the test balls continually submerged. The water is then gradually heated at such rate that the boiling point should be reached in about 1¼ hours. The water should be continually boiling and the test balls are to remain therein for a period of 6 hours unless in the meanwhile they show signs of distortion which naturally would render further testing unnecessary.

A good quality of Portland cement will invariably stand the test and the test balls become tough and in every respect resemble stone; in some instances white streaks may be seen on the surface, indicating minute cracks filled with lime hydrate.

Test balls made of poor quality of Portland cement will either crack and soon thereafter disintegrate or may boil to paste during the test.

In all instances when Portland cement fails to give entirely satisfactory result in the boiling test, the result of the standard test made on pats must by all means be awaited before using the cement.

4. FINENESS.

Two sieves shall serve to test the fineness of the cement: one having wires 0.10 mm. (0.00394 in.) in thickness and containing 900 meshes per sq. cm. (about 76 meshes per lin. in.); the other made of wires 0.05 mm. (0.00197 in.) thick and having 4,900 meshes per sq. cm. (178 meshes per lin. in.).

The fineness of Portland cement should be such that when it is passed through the sieve having 900 meshes per sq. cm. the residue shall not exceed 10 per cent. and the portion of cement passing the above sieve passed through a sieve having 4,900 meshes per sq. cm., shall not leave a residue exceeding 35 per cent. Hence

the total material tested and passed through both sieves shall not leave a total residue exceeding 45 per cent.

Remarks.

As Portland cement is used in combination with sand and broken stone and since the strength of binding, toughness and imperviousness of the concrete increases with the finenes of grinding of cement (the coarser grains merely take the part of sand), we should insist on having cement of a finely pulverized quality as to which examinations should be made by means of the described combination sievings.

One hundred grams (3.52 oz.) of Portland cement should be taken for the test sievings; the sieving shall continue until the dropping of cement ceases completely, which can be determined by repeatedly sieving over a white sheet of paper.

It would be a mistake, however, to judge the quality of cement solely by the fineness, since a great degree of fineness in the case of soft and poor quality of cement is of greater frequency than in those of the better grade; moreover, should the pulverization of the latter be coarser yet the strength of binding is often greater than that of the finely ground poor grade cements.

5. STRENGTH.

The strength of binding of Portland cement should be determined on a mortar prepared of sand and Portland cement.

The standard mixture is three parts of standard sand and one part of Portland cement, taken by weight.

The examination shall consist of tests for both tension and compression. The test pieces in each respective class shall be of the same shape and dimension and the test must be made by means of well constructed apparatus.

Test for compression shall be made after 28 days of hardening. The uniform good quality of material shall be controlled by means of tests for tension after 7 and 28 days of hardening.

The tests for tension must be made on test pieces, Fig. 1, the area of the strained plane of which must be 5 sq. cm. (0.775 sq. in.) [2.25 cm. (0.886 in.) long and 2.22 cm. (0.873 in.) wide]; tests for compression must be made on test pieces having an area 50 sq. cm. (7.75 sq. in.) [length of each side 7.07 cm. (2.78 in.)].

After the test pieces are prepared they must be kept in air for 24 hours and thereafter in water until tested.

Remarks.

Inasmuch as Portland cement is almost exclusively used in combination with sand, its binding quality must also be determined in a similar mixture.

Three parts of sand and one part of cement renders a convenient preparation whereby the quality of Portland cement may be satisfactorily determined.

Experience shows that the strength of concrete is materially affected by the chemical and physical properties of the sand used, therefore, in order to obtain good comparative results the sand used in the preparation of each and every test piece shall be of the same grain, weight and quality.

Such standard sand is obtained by first thoroughly washing a pure quality of silicious sand and, after drying, passing it through a sieve

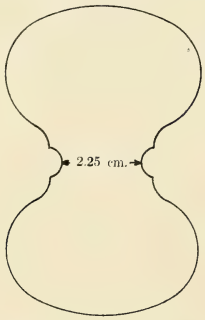

FIG. 1.—FORM OF TENSION TEST PIECE.

having wires 0.40 mm. (0.0158 in.) diameter and 64 meshes per sq. cm. (20 meshes per lin. in.), thereby removing the coarse pieces; then passing the sand through a sieve having wires 0.30 mm. (0.0118 in.) diameter and 144 meshes per sq. cm. (30 meshes per lin. in.), removing the finer grains.

The residue of this sieving will render the standard sand.

The tests for compression are standard tests because in practice the concrete is mostly subjected to compressive stresses. The ratio of compressive strength to tensile strength varies with the different cements and definite deductions cannot be made as to compressive strength from the tensile strength of a cement. Tests for compression must be made on test pieces after 28 days of hardening, because the quality of some cement is not sufficiently evident until after that period.

It is advisable to extend the time limit whenever possible, for it frequently occurs that while some cement may attain but a limited degree of strength at the beginning, later it will not only equal but exceed the toughness of other cements.

The uniformity of quality of shipments of Portland cement may be determined by means of tension tests; 7 and 28 day test pieces should be used for these determinations. The 7 day test pieces are used in order that the relative value thereof may be compared with the results obtained for compression on the 28 day test pieces and by using such relative values as a basis, such other data may be obtained as will enable a deduction of standard value for toughness. The 28 day test pieces used for tension give information in regard to the rate of setting.

In order to obtain a better comparison of Portland cement with other varieties of cement, it is advisable to test the tensile strength of the neat cement because some cement when in a pure state will render a better account of its peculiar qualities.

In connection with tests for toughness, the weight per liter of the loosely sieved sand must also be noted in each case. The sand must be passed into a cylindrical vessel 10 cm. (3.94 in.) high, the volume being one liter; the sieve used has wires 0.40 mm. (0.0158 in.) thick and 64 meshes per sq. cm. (20 meshes per lin. in.) and is held 15 cm. (6 in.) above the top of vessel during the operation of sieving.

The sieving should continue until base of the cone formed by the sand on top of vessel extends out to the rim which should thereafter be carefully smoothed off with the aid of a straight smoothing iron.

During the entire process shaking the liter measure should be carefully avoided.

6. TENSILE AND COMPRESSIVE STRENGTH.

The standard mixture for Portland cement (1 part cement and 3 parts standard sand, both by weight, 28 days of hardening, first 24 hours in air, 27 days in water) must have a tensile strength of 16 kg. per sq. cm. (227.6 lbs. per sq. in.); 160 kg. per sq. cm. (2276 lbs. per sq. in.) in compression.

Of six tests the average of the four best ones shall be taken as the result.

Remarks.

1. PREPARATION OF TEST PIECES.

(a) General Remarks.

Compression test pieces must be made by machine while tension test pieces may be prepared either by machine or hand. In disputed cases the results of the machine made test pieces shall be the criterion.

For testing the toughness of cement six test pieces must be made for each period of hardening.

The sand and Portland cement must be thoroughly mixed when dry. The sand is first weighed and deposited in vessel and the cement placed on top. These are intimately mixed with the aid of the trowel and then water is poured evenly over the entire surface and the mortar mixed from 1 to 3 minutes according to time limit, but by all means until intimately mixed. The mortar is then placed into well cleaned and moistened molds resting on metal plates; the subsequent depositing of remnants of mortar into the molds must not be permitted.

The test pieces must by all means be ready before the initial set develops, therefore, with rapid setting cements in particular, special care must be exercised in the preparation of the test pieces.

The locks on the molds used for tension test pieces should be of such form as will thoroughly resist the pressure to which the test pieces are subjected when being prepared; should the locks become loose the area of cross section to which the stress is applied becomes larger and consequently the result of the tests would be incorrect; therefore, the method of connecting the two parts of molds by means of springs should be rejected.

(b) Preparation of Test Pieces by Machine.

To obtain correct comparative results between compression and tension tests it is necessary to prepare the mortar of the same consistency and compactness. This result is obtained by adding the same quantity of water and mixing the dry materials in identically the same manner in each case.

The best means for determining the proportion of water is by taking the mixed dry material 780 g. (27.4 oz.) in weight and arbitrarily selecting the quantity of water, thoroughly mixing same, being careful that for quick setting cement the mixing should be finished within 1 minute; for slow setting cements, within 3 minutes; immediately after mixing the mortar is placed into the mold, compacting it by applying 150 blows of the 3 kg. (6.6 lb.) ram dropped from a height of 50 cm. (19.7 in.).

Should a slight watery slime appear on the surface of the test piece after it has been subjected to 120-130 blows of ram, the same indicates that the quantity of water used was a correct proportion; should this not be the case then the experiment must be repeated until the above result is attained.

The amount of water thus determined and expressed in per cent. will render the mortar of normal consistency and all test pieces shall be prepared accordingly.

Each test piece must be prepared separately; the quantity of dry materials for compression should be 780 g. (27.4 oz.), that for tension

168 g. (5.9 oz.) to be mixed with proper percentage of water and as described above.

The mortar is then deposited in the molds; the test pieces intended for tension shall be rammed with 150 blows of the 2 kg. (4.4 lb.) ram dropped from a height of 25 cm. (9.8 in.); those intended for compression with 150 blows of the 3 kg. (6.6 lb.) ram dropped from a height of 50 cm. (19.7 in.). Immediately after the last blow of the ram the plug and upper portion of mold is removed, the excess of material cut away and smoothed off and when hard set develops the test piece taken out.

The appliance serving for compacting test pieces must have a solid footing; it should preferably rest on masonry.

(*c*) *Preparation of Test Pieces by Hand.*

Each test piece should be prepared separately.

For each test piece 42 g. (1.5 oz.) Portland cement and 126 g. (4.4 oz.) of sand are thoroughly mixed dry in a vessel and a quantity of water, equal to about 10 per cent. of the dry materials by weight, is added so as to give the mixture the consistency of putty. With the aid of a trowel 35 cm. (14 in.) long and weighing 250 g. (8.8 oz.) the mixture is placed into the mold resting on a sheet of metal or glass plate in such manner that the mixture overtops the vessel; it is then tamped into the mold, using for the purpose the flat surface of the trowel, which should be 3 cm. (1.18 in.) wide and 5 cm. (1.97 in.) long. The tamping at the beginning should be gently around the sides and then with increased force until water appears on the surface of the mixture.

The ramming must by all means be continued until water is forced to surface of mixture, which requires about one minute. The material extending above the mold is then cut away and the upper surface of mortar smoothed off.

After the development of hard set the molds are carefully removed.

As a comparison of the quality of hand made test pieces, each series should be weighed both on removal from the water as well as on removal of molds, the average of the two weighings should also be noted in connection with the results of the experiments. The weights of test pieces of the same series should be about the same and not be less than 150 g. (5.3 oz.). Increase in weight in 28 days due to absorption of water shall not be more than 3 per cent. Test pieces lighter in weight or which due to absorption of water, reveal a greater increase in weight than stated, should be rejected.

(*d*) *Preparation of Tension Test Pieces of Neat Cement.*

The mold should be slightly oiled on the inner surface and placed on a sheet of metal or glass plate. One hundred and sixty-eight grams (5.9 oz.) of Portland cement with 35 g. (1.23 oz.) of water should be thoroughly mixed as long as the initial set (approx. 5 minutes) will

permit. The mold is then filled well over the top with the paste. Then proceed as prescribed for test pieces made of standard mixtures.

The molds should be removed only after the paste has sufficiently hardened.

Quick setting or very finely pulverized Portland cement will necessitate the use of a larger quantity of water in order to allow ramming to proper compactness; when the results of these test pieces are reported the amount of water used in preparing them should be stated.

2. STORAGE OF THE TEST PIECES.

During the first 24 hours the test pieces should be kept in air; to prevent uneven drying out they should be placed in a damp closet and after the period stated should be deposited and left in water until the time of testing.

The water in which the test pieces are deposited should be renewed weekly during the first four weeks; there should be sufficient water to keep the test pieces continually submerged. If the test pieces are stored for a long period a fresh supply of water should replace the portion evaporated.

3. MAKING THE TESTS.

The test must be made immediately upon taking the test pieces out of the water. For tension and compression 6 tests shall be made, each taken from the respective series of preparation. The average of the four best ones in each case shall be taken as the result of the test.

Since the rate of application of stress in tests for tension affects the result, the gradually applied stress shall be at the rate of 100 g. (0.22 lbs.) per sec.

When placing the test pieces into the clip care must be taken that the applied force should act in a direction perpendicular to the plane of the strained area, *i. e.*, it should be well centered and not be subjected to cross strains; the best means to attain this result is by having the contact points of the clips slightly curved and not straight, so that the test pieces have only two points of contact.

With a view to uniformity in results of compression tests the pressure should always be applied on the sides perpendicular to the direction of ramming and never on the rammed surface and side opposite.

The methods and results of the Cement Testing Station at the Royal Joseph University of Budapest shall be the standard in all cases of dispute.

D. Appendix.

The Resolutions governing the examination of different cements aim to make a proper comparison between, as well as to determine the respective values of, the various cements on a uniform basis.

In order that the best possible results may be secured in the use of cements, the following rules should be observed.

1. General remarks.

The following rules and data and instructions refer to such cements as fulfil in every respect the conditions laid down in the Resolutions. Any cement lacking but one single requirement of Resolutions must be rejected as not suitable for building purposes.

To avoid errors of all sorts, builders shall be required to designate all materials with the terms given under A; such ambiguous terms as "lime-cement," "hydraulic cement," "watertight cement," etc., must be avoided.

Further it is necessary that the builder specifically state in his order the kind of cement desired—whether quick or slow setting Roman or Portland cement—so that the forwarding agent may fully satisfy such requirements.

Such hydraulic cements as are in the grade of slow setting Roman cement shall be tested as per directions contained in the Resolutions for Roman cement; on the other hand in tests for the majority of slag cements particularly those which (as regards toughness) are in the same grade as Portland cement, the Resolutions relating to Portland cement shall be followed.

The examinations for such other cements as are but seldom employed in practice, shall be carried out in accordance with one or the other Resolutions, depending on the nature of the cement and the prospective application thereof.

As has been said in the Resolutions, the standard toughness of cement shall be determined by means of the 28 day tests for compression. Notwithstanding the fact that large shipments of cement may be arriving from the same place and awaiting examination, yet the result of the 28 day tests for compression must be

fully ascertained before putting any of the cement to use; in the meanwhile a test for tension must also be made on the 7 day test piece. If the result of the 28 day test has come up to the requirements, then all further shipments may be examined on the basis of the 7 day tests for tension since this particular relation between the two tests usually remains constant for cements delivered from the same locality.

2. SAMPLING MATERIALS FOR TESTS.

It is quite important in the case of large shipments of cement that the sample shall represent a fair average quality of the shipment.

For this reason the quality of cement necessary for the tests shall be procured as follows: From shipments up to 100 meter tons (22,000 lbs.) the sample shall be procured from 1/10 of the total number of barrels or bags; up to 1,000 meter tons (220,000 lbs.) it shall be obtained from 1/20, and from shipments of over 1,000 meter tons the material shall be gathered from 1/30 of the total number of barrels or bags.

Care should be exercised that the sample should be taken from as near the center of bag or barrel as possible, but by all means the sample taken should be at least 10 cm. (4 in.) from the outer surface of bag or barrel.

The cement thus gathered should be well mixed, and since it is usually shipped a considerable distance for examination, it should be packed in wooden boxes or glass vessels in order that it may be well protected from dampness; the packages should be plainly marked with name of sender and with name of town from where sent.

In accordance with the Resolutions the amount of cement required for the examination is 5 kg. (11 lbs.). If in addition to standard tests other investigations are also desired, such as toughness attained by proportionate mixing, efficiency of cement as regards withstanding frost, resistance to friction, etc., the quantity of cement shipped shall be 50 kg. (110 lbs.).

The sand used in these examinations shall be a pure quality of silicious sand, as described in the Resolutions. Should these examinations be of concern to the public, then the sand used in

tests shall be obtained from the Cement Testing Station at the Royal Joseph University of Budapest.

3. COMPARATIVE VALUES OF CEMENTS.

The practical value of such cement as fulfils the requirements of the Resolution shall depend on its capacity for sand. All things being equal such cement is to be regarded as having the maximum value which will meet the requirements of the 28 day test for compression with the greatest amount of sand added thereto or vice versa, such cement has the maximum value of which the least quantity is required to be mixed with sand and at the same time produce the above toughness.

The total weight of cement (obtained in accordance with the above) multiplied by the price of the cement when delivered to work, will render the monetary value of the cement at the location of the work.

On extensive construction work it is also advisable to determine the value of the sand used.

4. PROPORTIONS FOR MIXING.

In construction work cement as a rule should be used in combination with sand. The proportion of sand shall primarily depend on the value of cement as outlined above; secondly it depends on the special object to be attained with the mortar.

Should the mortar be intended merely to withstand stress, the proportion of sand in that event should be fixed so that the resulting mortar shall satisfy such conditions; if, however, in addition to strength it is also necessary that the concrete or mortar be as nearly impervious to water as possible, then the mixture must be prepared dense. It must be prepared in such way that all the voids in the sand may be filled up with hardened cement when final set develops in the concrete.

This principle shall hold good for all cases, whether the mixture is to be used merely for plastering purposes or as a binder for artificial stones or when used for concrete. If in the latter case besides strength imperviousness is also essential, then the voids between the gravel must be filled up with such impervious matrix, *i.e.*, each gravel must be entirely surrounded with such impervious mortar.

It is evident that the toughness and density of the gravel used in the making of the concrete must at least equal the density and toughness anticipated for the mortar.

The proportion of sand for mortar or the proportion of sand and gravel for concrete shall be expressed by volume, while the quantity of cement necessary to gain the special object shall be expressed by weight.

The correct proportion shall be determined by ascertaining how many cubic meters of sand and gravel and how many kilograms of cement should be taken per cubic meter of concrete, or how many cubic meters of sand and how many kilograms of cement should be taken per cubic meter of mortar.

In practice it shall be permitted that the proportion of cement, as determined by weight, may be measured out by volume.

(*a*) *Proper Proportion of Materials for Concrete of Certain Strength.*

The strength of the mortar depends not merely on the toughness of cement used, but also on the size of grain of sand and its other physical properties.

It is therefore impossible to lay down general rules fixing the proportion of the materials for concrete of certain strength; in important cases the question as to when a certain kind of sand mixed with various proportions of particular cement will produce concrete of what strength, must be settled by experiment.

In order to fix the proportion of the materials which will produce concrete of certain strength, there must first be determined the amount of voids contained in the gravel or broken stone, which gravel or broken stone have a certain minimum strength. These voids must all be filled with a matrix of certain required strength, and the quantity of matrix determined upon as necessary to fill up such voids must be increased by from 15 to 20 per cent., depending on the size of the gravel or broken stone, in order that not merely the voids may be filled, but that the entire surface of each and every gravel or broken stone may be imbedded in matrix.

The volume of vessel used for determining the amount of voids should be as large as possible; it should not be smaller, however, than 0.125 cu. m. (4.4 cu. ft.), which is filled to the brim with the

proposed gravel or broken stone. Now water is poured into the vessel until (having filled all the voids) the water reaches the surface. The increase in weight in kg., due to the addition of the water, equals the volume of the total voids in liters.

It is to be noted that the calculated strength of concrete will be attained only if the strength of the gravel or broken stone equals that of the hard set matrix. Should the strength of the gravel or broken stone exceed that of the matrix then the strength of the concrete will also be greater; the extent of such increase in strength cannot be stated at present, since the necessary experiments are as yet lacking.

The proportion of material thus fixed shall be valid for concrete used in dry construction, and its preparation must in every respect correspond to that of the matrix examined for strength, which has served as a basis in fixing the proportions of the materials, *i.e.,* it must be prepared with the same proportion of water and should be compacted by the same method.

If the concrete is to be incorporated in the structure without tamping, then the test pieces should be prepared by merely shaking the concrete; if, however, the concrete is to be tamped, then the test pieces must be made correspondingly compact.

If, for example, it is desired to produce such a concrete, which —having been tamped—after 28 days of hardening should have a compressive strength of 80 kg. per sq. cm. (1,138 lbs. per sq. in.), then there must first be determined the proper proportion of materials which when placed in cube form and tamped by hand shall, after 28 days of hardening, withstand the desired compression of 80 kg. per sq. cm., *i.e.,* the volume of sand and the weight of cement required to produce a hard set concrete having the stated strength must be determined.

Then there is determined the volume of voids per cubic meter of the gravel used by increasing the obtained volume of matrix by 10 to 15 per cent., and thus arriving at the proper proportion of matrix for the concrete.

Adding the volume of this matrix to the real volume of the gravel (excluding voids) the result will be the quantity of concrete that may be gained from 1 cu. m. of gravel; the answer naturally is greater than 1 cu. m., and the volume of the gravel and of the

matrix required for 1 cu. m. of concrete must be fixed from these values. Knowing the amount of matrix required for the structure as well as the mixing proportions, there can then be determined the volume of sand and the weight of cement necessary for 1 cu. m. of concrete.

Whenever it is deemed more convenient to fix the mixing proportion of cement by volume, then the cement of the required weight is placed in a suitable vessel in such loose state as is customary to place the cement in the measuring boxes on the work, and the volume thereof is determined.

By thus fixing the proportion of cement by volume and taking it as a unit the quantity required of each material by volume is determined.

The advantage of these calculations is that by weighing each individual box of cement there is more certainty that the correct proportion of cement is being used.

(*b*) *Determination of the Proportions for Dense Concrete.*

Such concrete may be considered dense wherein the proportions of the materials intended to fill up the voids are fixed so as not merely to fill up the voids, but that the void filling material should in a certain measure be in excess.

In dense concrete the voids in the gravel or broken stone are filled up by the mortar. The cement used in such mortar is of such quality that in filling up the voids between the grains of sand the cement itself becomes perfectly dense when hard set develops.

Therefore, first two things must be determined: viz., the proportion of water necessary to a unit weight (1 kg.) of the proposed cement and also the volume (when hard set) of the resulting mixture having the desired density when completely hardened.

Although the method for determining the volume of the hardened cement is by adding the volume of the water used to the real volume (excluding voids) of cement, it is better practice to determine these values by direct experiment in each case, because by doing so there is simultaneously ascertained the degree of density of the hardened cement.

The determination of these volumes is most conveniently made by means of the Schuman volumenometer.

This apparatus consists of a flask, either a low cylindrical one, the upper portion being conical, or one having a spherical shape with flat bottom; the volume of it is about 200 cu. cm. (122.3 cu. in.). It is furnished with a hollow glass stopper which extends into a glass tube divided into cubic centimeters. A pure quality of turpentine is put into the flask, reaching to slightly above the zero (0) mark at the lower end of glass tube. The upper part of glass tube is funnel shaped. Through this opening is introduced the weighed cement or hardened and powdered cement, the turpentine in flask will then rise to a point indicating the real volume (excluding voids) of the material introduced; the reading must not be made, however, until the liquid is perfectly clear and free from air bubbles.

The voids in sand are ascertained as described under (a) for determining the voids in broken stone; in the case of the sand, however, a certain amount of sinking is observed when the water is poured over it. The determination is made by pouring water into the weighed sand filled vessel until it reaches to the brim. The increase in weight reveals the amount of voids in the loose sand; upon allowing the vessel to stand from 12 to 24 hours it will be observed that the sand settled and a certain amount of water is visible above the sand. This quantity of water should then be removed in some convenient manner (by suction tube) and the difference in weight renders a volume equivalent to the volume which the sand settled.

The sand used in the preparation of concrete naturally comes in contact with water, and it can be assumed that the sand settles to its smallest volume, hence when making the determinations for the proportion of cement, the amount of voids to be allowed for in sand should be for such settled sand as is described in the preceding paragraph; this is obtained by deducting from the total voids in the loose sand the volume to which the sand settles during the mixing.

There is ascertained, therefore, the volume say 1 cu. m. of sand will have when it is settled and also the proportion of cement required (together with the additional percentage in excess) to fill the voids in the settled sand; the sum of these two quantities gives the amount of mortar that can be gained from 1 cu. m. of sand; the volume obtained is always less than 1 cu. m.

The amount of voids in 1 cu. m. of stone may be ascertained by the method already described and the proportion of mortar (with the additional percentage in excess) necessary to fill the voids in the stone can be determined; the volume of this mortar added to the real volume (excluding voids) of the 1 cu. m. of stone gives the quantity of prepared concrete that can be obtained from 1 cu. m. of stone; the value of this is always greater than 1 cu. m. The volume of stone and matrix necessary for 1 cu. m. of prepared concrete must be determined from this data. Having determined the volume of concrete required, the volume of sand and weight of cement necessary to its preparation are ascertained therefrom.

Moreover, having determined the weight of the required proportion of cement and thereafter ascertaining its volume by means of the method already described, there is obtained the mixing proportion by volume.

Concrete mixed in accordance with the calculated proportion will by all means be dense. The question, however, as to whether it has the required strength must be settled by examination made of the test pieces prepared from the concrete.

Committee appointed for drawing up Resolutions Governing the Uniform Examination of Cements:

SCHUSTLER, JOZSEF, *Secretary.* NAGY, DEZSO, *Chairman.*

BEKE, JOZSEF, ERDOS, FERENC, STRICKER, SANDOR,
KISS, ISTVAN, LANDAU, GUSZTAV, KADAR, GUSZTAV,
MIHALYFI, JOZSEF, SZIVAS, GYULA,

Adopted by the Hungarian Society of Engineers and Architects at a regular meeting (385), held January 16, 1897.

FITTLER, KAMILL, *Secretary.* CZIGLER, GYOZO, *President.*

THE MEASUREMENT OF CONCRETE CONSTRUCTION.

By Robert A. Cummings.*

Reinforced concrete is composed of a large number of elements, all of which must be skillfully brought together to complete a perfect structure. In order to accomplish this each element is carefully considered, specified and combined with much detail in execution. It not infrequently happens that when we are engaged in such detailed work, the broader view of our occupation is lost sight of, this brings me to the point to which I desire to direct your attention, viz., the Measuring of Concrete.

So far as is known there are no rules governing the payment by fixed units of measurement for concrete work. Each contract seems to be a law unto itself. Certain trade customs in Chicago have been recently incorporated into a set of methods of measurement which to my mind are most reprehensible, because they violate the principle of honest and square treatment for all interests; yet I fully appreciate the difficulty of satisfactorily providing adequate methods of measurement to meet all conditions. It seems to me that this Association should take immediate action to prepare suitable rules to govern the measurement of concrete work.

It might be well to illustrate this necessity. For instance, if this Convention Hall were constructed in concrete how would the contractor be paid for the arches, ornamental columns, niches, balcony railings, etc.? Should it be by the square foot of surface or the volume of concrete? Again it is not an uncommon practice for a certain class of Engineers and Architects to deduct the volume of space occupied by the reinforcement from the mass of the concrete, and I have heard of bolt holes and very small openings being deducted. This I think is sufficient evidence to warrant consideration of the above suggestion.

* Consulting Engineer, Pittsburgh, Pa.

THE USE OF COMPRESSED AIR IN HANDLING MORTARS AND CONCRETE.

By G. L. Prentiss.*

In analyzing the title of this paper it is evident that a consideration is to be given first to mortars and concrete themselves, secondly to compressed air as a motive power and thirdly to the handling of the concrete and mortar by means of compressed air. For the purposes of this discussion, mortar may be defined as a mixture of lime, cement or other cementitious material with sand and water; concrete as a mixture of cement, sand, water, and agglomerate material such as stone, cinders, etc. Compressed air will have consideration as a motive power.

The handling of concrete and mortar by means of compressed air comprehends within the term "handling" not only the preparation of the concrete and mortar from the raw materials, but also the placing of the product in position upon the work. The logical treatment of this subject will of course involve some preliminary statement concerning its history.

As mortar and concrete are the result of mixing various materials together, the history of the art is largely that of the various attempts to improve this mixing as to quality of product and speed in obtaining a satisfactory mixture. The materials, when mixed, have, however, only passed through half of the process of making them useful, the consideration of remaining element is equally instructive, namely the historical study of the attempts that have been made to improve the methods of placing the materials in position upon the work.

There are, of course, four possible methods of handling the materials. Hand labor throughout, involving the mixing by hand and the placement of the work by hand; machine mixing and hand placement; hand mixing and machine placement, and lastly, machine mixing and machine placement.

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It does not seem necessary to discuss hand work. The machine mixing of mortar and concrete is also a subject with which all are familiar. The art has been carried forward to a condition of high efficiency and great capacity by the development of the various concrete mixers of commerce. In none of these machines, however, has compressed air been used for mixing material so far as I am aware.

The placement of concrete and mortar by machine is that division of the subject where the application of compressed air really begins to find a place. The history of the art in this connection is a very short one. The subject seems to have had attention by engineers only within the past few years.

In 1906, on the Paris-Lyons-Mediterranean Railway of France, the engineers were confronted with a problem of grouting with cement and sand the outer surface of arches of concrete which formed the inner lining of a railroad tunnel. The arches were being affected by dampness, were leaking water into the tunnel and were rapidly deteriorating. The engineers met this problem by using a machine consisting of a charging hopper connected with a pressure tank, which tank also received from a compressor air under pressure of 40 or 50 lbs. per sq. in. Having charged the pressure tank with the grout composed of cement, sand and water, the pressure which was then put upon it forced the materials through a line of flexible hose to a discharge nozzle which was applied to holes drilled through the arch of the tunnel. The grout was forced through these holes and allowed to flow over the outer surfaces. The results from an engineering standpoint were pronounced satisfactory, but the machine gave a great deal of trouble, as it clogged with the grouting constantly and the hose choked up. The process was therefore very tedious and expensive.

In 1909 on the Metropolitan Railway of Paris, M. Suquet grouted a tunnel with the same object as in the case last cited but used a centrifugal pump to give pressure. The work was accomplished in spite of many break-downs and at great cost. In the same year and for the same purpose a grouting pump was used, but in this case, no compressed air was made use of in handling materials.

In 1909, in this country, Messrs. J. W. Buzzell and William

H. Larkin, Jr., undertook work on somewhat similar lines to meet a situation of extraordinary difficulty. They used a hopper to receive a mixed concrete of 1 part cement, about 3 parts sand and 5 parts 2-in. broken stone mixed in the usual manner with water. The mixed materials were received into the discharging hopper which then led to a pressure tank and by the shutting of the appropriate valves were put under pressure and the concrete was forced through a 4-in. pipe main to the site of the work. At 80 lbs. pressure it was found the material could be conveyed some 400 ft. High pressures were found to be disadvantageous and a pressure of 40 to 50 lbs. was much more efficient in operation. Even at that pressure the materials when delivered at the discharge end of the main had to be slowed down by being delivered into a receiving box and from it allowed to fall upon the work by gravity.

Compressed air is used commercially in another class of machine of a somewhat analogous character but not strictly within the limits of this subject. This is what is known as the whitewash or paint machine.

About three years ago, Mr. Carl E. Akeley, at that time in charge of the Department of Taxidermy of the Field Museum of Natural History at Chicago, Ill., in his early experiments along these lines constructed a machine for placing plaster by the use of compressed air and evidently got his idea in the first place from these same whitewash machines. He mixed the materials in liquid form and applied compressed air, driving them through a length of hose to a nozzle and from it to the work. The experiment was not successful. When the materials were wet enough to flow, they were too wet to stay upon the wall.

It will be appreciated from what has just been said that all these machines for the handling of the placement of concrete and mortar have been developed by engineers of enterprise and original mind for the accomplishment of special work of extraordinarily difficult character. The obstacles were otherwise insurmountable and the commercial aspect of the work was secondary in importance to the accomplishment of the end in view.

Lastly, it may be mentioned that along the specific lines of this discussion, a grouting machine has been in use for a con-

siderable period by a prominent firm of paving contractors. This machine operates along somewhat similar lines using cement grout mixed with water and forcing the materials through a grouting nozzle between the blocks of stone pavement.

Thus the history of the art with regard to the mixing of and placing mortar and concrete by means of compressed air dis-

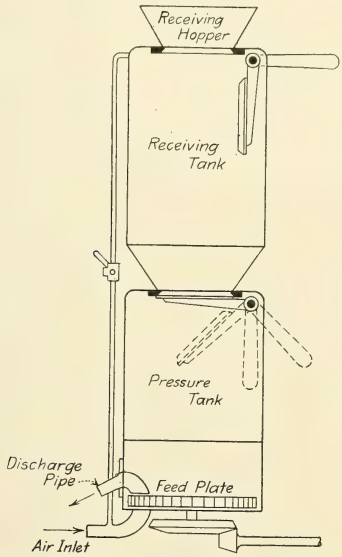

FIG. 1.—DIAGRAMMATIC SKETCH OF CEMENT GUN.

closes successful machines for the mixing of concrete and the application of the same by hand, but in the consideration of machines for placing the concrete, no such success can be claimed. That part of the process when conducted by machine has proved too expensive, it is subject to constant interruption due to the setting of the materials in the conveyor pipes and in the machines themselves and the work has been very inferior. This matter of placing concrete by machine has therefore not shown anything

FIG. 2.—CEMENT GUN NOZZLE IN OPERATION.

like the efficiency or the commercial success of the mixing machines.

No machine has been developed until a very recent period which both mixes and places the concrete or mortar. The desiderata in mixing and placing these materials are evidently,

1.—Cheapness of operation.
2.—Certainty and continuity of operation.
3.—Flexibility of operation.
4.—Homogeneousness of structure.
5.—Maximum density.

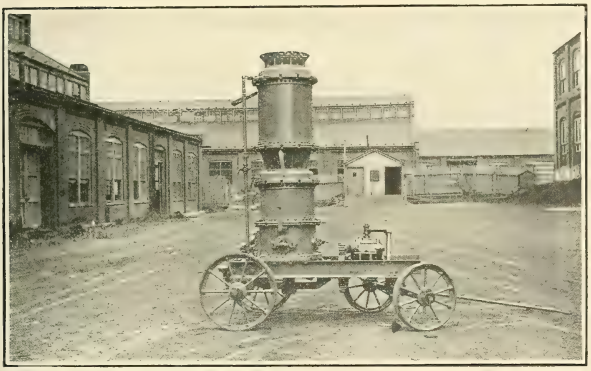

FIG. 3.—CEMENT GUN UNIT ASSEMBLED ON WHEELS.

All the machines mentioned above have evidently attempted to cover some of these points, but it is certain that except as to the whitewash machine, none have become commercially prominent. The conclusion seems inevitable that none of the machines have filled even approximately all these desired conditions.

There has recently been developed, however, a machine which seems to meet all the requirements and an attempt to describe this machine and its operation is the remaining purpose of this discussion. In contradistinction to all previously developed machines, it mixes the materials dry and handles them in a dry condition up to the moment of their placement upon the work.

It is able to operate continuously and therefore is a commercial machine. It has a very wide range of work. It is capable of both mixing and placing concrete or mortar. Its concrete is of most extraordinary density and is thoroughly mixed and thoroughly homogeneous. The mixing and placing of the concrete is extraordinarily cheap and it employs compressed air as a motive power. This machine is commercially known as the Cement Gun.

Proceeding to a further description of the machine itself:—For mortar or for concrete where stone or cinders are not to be

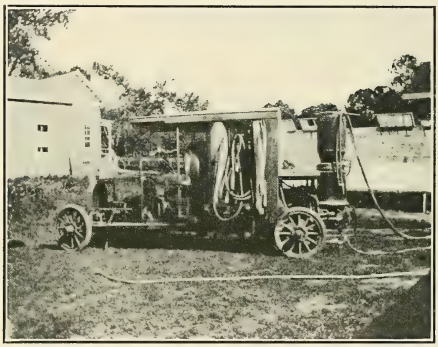

FIG. 4.—AUTOMOBILE TYPE OF APPARATUS WITH COMPLETE POWER PLANT.

used (including in this description all stucco, plastering work and similar construction), the materials are mixed in a dry condition either by hand in the usual way or by a cube mixer; thus at the outset it will be seen that a wide variation in use is contemplated, for it is evident that for small contracts, an elaborate mixing plant is not necessary, while for large contracts it is quite indispensable.

When the dry materials have been mixed together as above described, they pass from the mixer either by hand or by means of conveying buckets to a point above the receiving hopper of the machine itself. This hopper, Fig. 1, leads to a riddle and the materials pass from the riddle into the upper tank which can be

hermetically closed. Directly beneath the upper tank is a similar lower tank of the machine. These two tanks may be called respectively the Receiving Tank and the Pressure Tank. At the bottom of the pressure tank is the discharging mechanism.

Having thus briefly described the machine, the process of handling from the time the materials reach the machine is as follows: As the dry materials pass through the hopper and riddle they fill up the receiving tank to a quantity equal to approximately one barrel of material. The whole is then sealed and the charge is dropped into the pressure tank, whereupon the receiving tank is

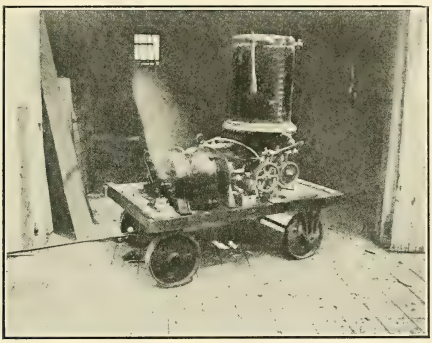

FIG. 5.—CEMENT GUN WITH MOTOR DRIVE, MOUNTED ON TRUCK.

ready for another charge. When the material has reached the pressure tank, it falls on the discharging mechanism, consisting of a flat circular slotted plate revolving at a uniform speed and delivering with each slot a charge of material over the air admission pipe. The air pressure causes the material to move upward into the discharge pipe leading through the hose to the nozzle.

It should be borne in mind that during all this time the materials are dry finely comminuted particles of sand and cement or similar material. As this action goes on in the pressure tank the materials are deposited in such a manner as to cause a certain amount of material to flow through the outlet opening from said pressure tank. There is thus a constant discharge of material

driven by the force of the compressed air out from the machine and along the line of hose which leads from the machine to the site of the work.

At the outer end of said hose there is a nozzle, Fig. 2, which receives the dry materials and by a series of ingeniously contrived parts, administers the required amount of water at the

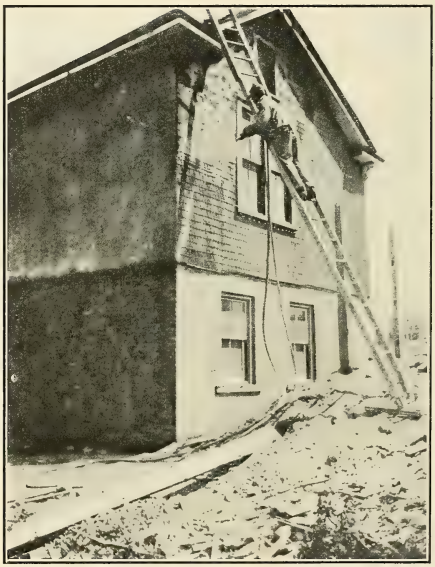

FIG. 6.—NOZZLE IN OPERATION APPLYING CEMENT STUCCO.

outlet end of this nozzle. This water when thus added to the dry materials is by the air pressure aspirated out of the end of the nozzle and driven with force against whatever may be the required emplacement of the materials. The water is led by a separate hose from a water pump and is controlled by a valve upon its own line which forms a part of the nozzle, which may be easily held in one hand and is manipulated just as the nozzle of an

ordinary water hose would be. Thus the dry, finely divided material and the atomized water are together and at one and the same time discharged with force upon whatever position they are to occupy. When they land upon the work they are concrete.

The commercial Cement Gun is composed of the following parts:—The machine itself which has been described above; a suitable engine or motor for driving the whole machine; an air compressor for giving the required amount of air; a water pump for giving the required amount of water and the driving mechanism for connecting the various parts together. The machine is

FIG. 7.—OLD BUILDINGS AND FENCE COVERED WITH CEMENT STUCCO APPLIED WITH CEMENT GUN.

mounted on wheels so it may be taken from place to place, Fig. 3. In Fig. 4 is shown the automobile type of Cement Gun with complete power plant. This is the original machine used to repair the staff or lime stucco coating of the Field Museum at Chicago, and after continuous service for 3 years is still in efficient operation. Fig. 5 illustrates a Cement Gun with motor drive, mounted on a hand truck for use where compressed air and electric current are available from local sources.

It is essential in the operation of such a machine that a wide range of pressure and a wide range of speed should be available. This is done by simple but quite adequate means. The

whole of the operation of the machine is under the control of one man standing upon the platform. He superintends the charging of the dry materials, gauges the speed and the pressure at which the machine should work, and looks after the various parts of the apparatus. Another man, not necessarily especially trained, puts the material upon the work. In cases where finishing is necessary, a third man, who should be a competent plasterer, follows the nozzle operator and with the ordinary tools puts upon the work the finish required. It is also obvious that for special purposes the method of driving, the method of arranging the

FIG. 8.—REAR VIEW OF LABORATORY OF THE GENERAL CEMENT PRODUCTS COMPANY, WHITESTONE LANDING, L. I., CONSTRUCTED WITH CEMENT GUN.

compressors and pumps and the number of Cement Gun units made use of at one time are all the subject of arrangement and variation to suit the necessities of the case. Thus a machine has been put into commercial successful use which not only mixes mechanically the ingredients of mortar or concrete, but also places the same upon the work by means of compressed air.

In conclusion, it is fit that something be said concerning the character of the work. The desirable qualities of handling concrete above enumerated were cheapness; range of application; certainty and continuity of operation; homogeneousness of structure and high density.

A competent plasterer who was not especially familiar with the machine but had the opportunity to see it work, said that the machine, in his judgment, was equivalent to the work of from 5 to 8 plasterers with the attendant helpers and facilities for mixing the materials. In other words, the one machine would place mortar as rapidly and as completely as 5 to 8 men could do it. From other experience, it is quite clear that this man underestimated the capacity of the machine. At slow speed and 40 lbs. air pressure, the smallest of these units will place from 12 to 15 barrels per hour and at 5 sq. yds. 1 in. thick to the barrel, which

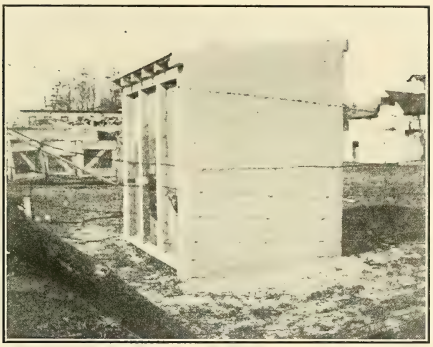

FIG. 9.—FIRE TEST HOUSE.

is a very moderate estimate, this would mean the placement of 1 sq. yd. per minute 1 in. thick, and this is ordinary practice for the machine. It is clear, therefore, that the machine must produce its results cheaply.

As to the certainty and continuity of operation: The first machine ever constructed was put into service some three years ago. It is still doing excellent work although it was a machine merely assembled and not especially well proportioned. The standardized machines which have been the result of the experience with the experimental ones are thoroughly competent to operate continuously for any length of time.

As to the flexibility of the operation—it is a little short of marvelous and will be referred to later on.

As to homogeneousness of structure, the ability to absolutely

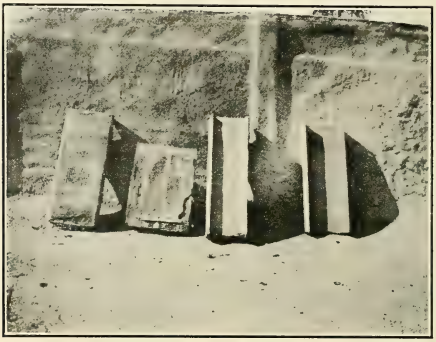

FIG. 10.—PIECES OF STRUCTURAL STEEL, WITH AND WITHOUT WIRE MESH, COATED WITH CEMENT GUN.

FIG. 11.—CEMENT GUN CONVERTING WIRE FENCE INTO CONCRETE FENCE.

regulate the proportions of the constituents necessitates a very even composition, while as to density, the fact that the machine does its work in the way described assures this in the very highest degree.

As to the work which the machine has done and is capable of doing, a few words may with propriety be said.

One of the great fields of usefulness is the coating of the outside of buildings, whether new constructions or old buildings of wood, stone or brick, with stucco concrete. Fig. 6 illustrates the application of cement stucco to an old building previously covered with wire mesh. Old wooden buildings and wire fence coated with cement stucco are shown in Fig. 7. The structures look like new ones, are externally fireproof, their life is prolonged and the annual expense of painting and upkeep is elimi-

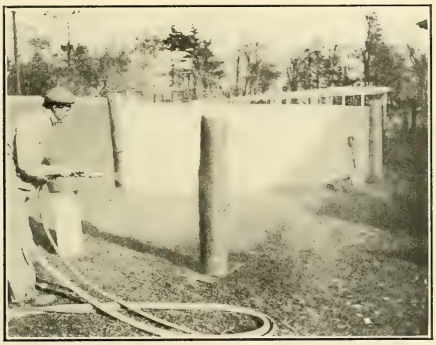

FIG. 12.—BUILDING UP A SECTION OF CEMENT PIPE FROM WIRE MESH SKELETON.

nated. Fig. 8 is a rear view of the laboratory of the General Cement Products Company, Whitestone Landing, L. I., constructed with the Cement Gun.

The advantages of such work are self-evident as to fire protection, freedom from future repairs, improvement in appearance, increase in the insulating ability against temperature changes and most important of all the great cheapness of this work. It can be accomplished at figures which actually put it into competition with ordinary paint. It is so dense that it is damp proof to a most remarkable degree.

Another great field is the coating of the inside of factories,

mills, warehouses, etc., as a protection against fire. It is an ideal means of securing slow burning construction at minimum cost. The fireproofing function of concrete thus applied has been very thoroughly tested and the Cement Gun concrete has stood the severest tests conducted by recognized experts with entire satisfaction. A small building of 2 x 4 in. wooden frame, covered inside and out with building paper, wire mesh reinforcement and 1 in. thick cement-sand stucco, leaving 4 ins. air space between interior and exterior walls, was erected for a fire test, Fig. 9. After the fire, scientifically built, had burned 16½ minutes,

FIG. 13.—ADAPTATION OF CEMENT GUN TO TREE SURGERY.

buckets of water were thrown through the door (not shown) against the inside hot walls and a little of the surface scaled off but not a crack developed. Insurance representatives were present at the test. The outer coating of rear wall was left off in order to determine whether or not wood studding would be affected by fire, but studding was not even charred.

The covering of steel columns and girders in skeleton steel construction has also been very successfully handled. Corrosion of steel has been thoroughly taken care of by this means and building constructors, engineers and architects have expressed themselves in the highest terms. Fig. 10 illustrates pieces of steel beams coated with cement-sand mortar for protection against

rust and corrosion; two pieces with and two pieces without reinforcement.

Walls of stucco concrete have also been constructed with remarkable success. On one occasion a structure, Fig. 11, was built of two fence posts placed about 12 ft. apart between which was stretched chicken wire fencing about 5 ft. high, a couple of wooden doors were placed behind the fence close to it and the

FIG. 14.—HUNTERS BROOK SIPHON OF THE NEW YORK WATER SUPPLY ON WHICH THE CEMENT GUN IS BEING USED.

concrete was placed with the Gun from the front of the fence against this matrix to a thickness of 2 ins., enclosing the wire fence, and thus building a wall 12 ft. long, 5 ft. high and 2 ins. thick. An hour after the last concrete was placed the doors were taken away. The wall stood perfectly and has shown no defect since. It will be observed that no forms were required and that the speed of construction was phenomenal.

Fig. 12 shows the building up of a section of cement pipe

using a wire mesh skeleton; whereas Fig. 13 illustrates the application of the Cement Gun to tree surgery.

Massive concrete is handled in the following manner: A steel framework is constructed and covered by a layer of fencing, thus forming a kind of casket. Into this the broken stone or cinders are fed in a dry condition and while being deposited they are constantly sprayed over with the cement, sand and water from as many Cement Gun nozzles as the character of the work

FIG. 15.—CEMENT GUN STATIONED ON SIPHON.

requires. The impacting of the concrete is thus very thorough and the density and strength very great. When the casket is entirely filled up the structural steel work is incorporated as a part of the reinforcing material by being coated over with an inch or so of stucco concrete and the job is complete. In all this work it is evident that no forms need be used. The work can be placed vertically from above or from below or horizontally; it is stronger than any other similar content.

A very important contract which is at present being executed by means of the Cement Gun consists in lining the siphons in con-

nection with the water supply scheme for the city of New York. The Board of Water Supply stated in the specifications that all the steel siphons were to be lined internally with a cement mixture to a thickness of 2 ins. A vast amount of experimental work was done with various types of collapsible forms, grouting machines, etc. Finally a test was made with the Cement Gun which fulfilled all the requirements, and the work which has been accomplished on the Hunters Brook Siphon, Fig. 14, has had the full endorsement of the engineers. This work necessarily has to be accomplished under very difficult conditions, and by means of artificial light. The siphons are 11 ft. 3 ins. in diameter and the total length to be treated in this manner is 13,663 ft. The Hunters Brook Siphon upon which the Cement Gun is at work is 1,491 ft. long and about one-half of this has already been completed, demonstrating the absolute practicability of this method. Fig. 15 shows the Cement Gun located on the siphon and the manhole through which the two lines of hose are taken to the interior.

This part of the discussion could be indefinitely extended, but to no special profit. The usefulness of such work in mine construction and in a number of other great fields has only to be mentioned to be appreciated and understood.

Finally, a word as to the reinforcing materials needed. The expensive and especial metal lath construction is entirely done away with and ordinary fencing wire is substituted for expanded metal, thus in house and building construction the costs are very greatly reduced.

This subject might be gone into at any length. Sufficient has been said, however, to give an indication of the great new industry now developing before the world, which is made a possibility only by the use of compressed air in handling mortar and concrete.

THE SPOUTING OF CONCRETE.

By Fred E. Engstrum.*

The universal adaptation of the Gravity System of conveying and distributing concrete is herewith briefly stated, and embodies for the first time all the latest engineering data and practice relative to the art. The Gravity System of conveying and distributing concrete originated in Southern California, spread gradually up the Pacific Coast, and then east. The patents of the appliances of this system are practically basic, and are covered by Letters Patent Nos. 948,719, 948,723 and 948,746. These are the first recorded patents of the art, and cover in general the use of a revolving mast or boom, supporting a trough or pipe, with swivel and movable trough at end of boom, the combination of which gives a horizontal motion in any desired plane at the delivery end of pipe or trough. There are numerous other patents pending, all more or less interfering with the above letters patent. They are combination patents, having little value without the use of the above basic patents.

Many licenses have been signed throughout the United States for work on buildings, dams, reservoirs, etc. These jobs range from 1200 to 100,000 cu. yds. each.

For the successful operation of the Gravity System, one of the most necessary things is the proper mixing of the ingredients. It is unnecessary to go into any extended argument relative to the nature of stone, sand, voids and the different proportions of same. Having given the proper proportion of the ingredients, it is absolutely necessary that enough water should be added to the conglomerate to produce a specific gravity such that the rock is held in suspension, the mixture will flow like a thick gravy with no sign of excess water; in other words, it should have a saturated amount of water and no more. This particular mixture is surprisingly slippery, and flows readily around all steel. It also has an oily nature which is due to the cement. The discovery of this particular

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nature or consistency was the cause of revolutionizing the existing methods of distributing concrete, and led to the development of this system.

Another great advantage in the conveying and distributing of concrete by the Gravity System is that large quantities can be handled by a small stream, placed where wanted, practically obviating all air pocketing, with absolutely no separation, and overcoming all tendency to laitance. The great freedom from any action of laitance has been noted particularly by engineers who have carefully watched the results obtained where this system was used.

On account of the concrete flowing in a small but continuous stream, this system is especially adapted to the pouring of thin walls and columns in which there is a large amount of reinforcement and it assures in every case practical, monolithic construction.

The operations of the Gravity System are as follows:

1. The mixing of the ingredients, stone, sand and cement in any suitable batch mixer.

2. Hoisting of the mixed concrete by skip to an elevated hopper.

3. Transferring of concrete through pipes leading from this hopper to the point where it is desired to be poured.

A few of the great points of superiority of this system over the wheelbarrow or cart method of distribution, are as follows:

1. The obtaining of the initial set of the concrete when in place, on account of the rapid handling of the mass and depositing the material into the forms in the shortest given time.

2. A uniform, homogeneous, plastic mixture, due to the large quantity which can be produced at one time, and the absolute overcoming of all tendency to laitance.

3. The unlimited variation of the design of buildings to which it is adapted, on account of the flexibility of the equipment from the simple foundation and dam work, to the most complicated dome, column and frieze work, such as found on tall buildings and overhanging cornices.

4. The use at all times of a standard equipment for the entire process of mixing, transferring and pouring, which can be installed at a minimum expense.

FIG. 1.—TYPICAL INSTALLATION OF GRAVITY SYSTEM. SPRECKLES OFFICE AND THEATER, SAN DIEGO, CAL.

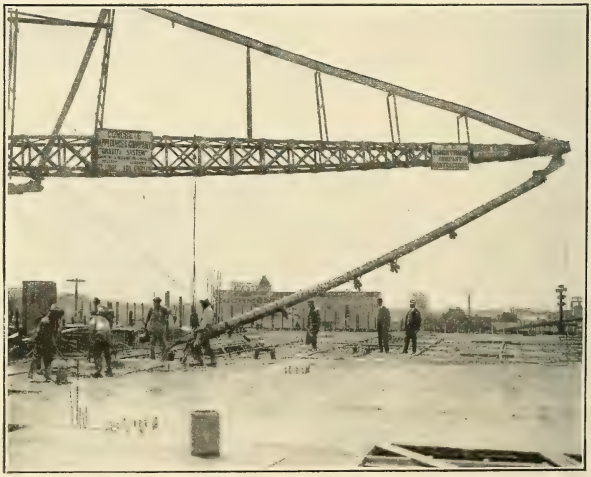

FIG. 2.—THE POURING OF CONCRETE.

5. The saving of breakage of hollow tile, and displacement of steel on floor work, also the saving of spilling of concrete and the cost of cleaning the floors and beams afterward, which is unavoidable when using the cart or wheelbarrow method.

6. The assurance of deep, narrow walls being perfectly filled.

7. The handling of large quantities, the output being limited only by the mixing and hoisting capacity of the plant.

The following are the instructions for operating the Gravity System:

1. Run about 10 ft. of water in skip.

2. Charge mixer, using mostly sand with cement, and plenty of water.

3. Run up skip with water and drop at once, following up as quickly as possible with charge of concrete.

4. Run mixture such that it has the consistency of a thick gravy, so that when it levels off, the rock is seen held in suspension; if, when dumped in skip, it stands up, it is too thick; again, if it levels off and shows 1 in. of water, it is too sloppy. The specific gravity of the conglomerate should be such that the rocks are held in suspension.

5. There should be a man in tower to operate the concrete gate on concrete hopper, which regulates the flow of concrete through pipe. Run a stream which fills about one-third of the conveyor pipe, and time discharge so that stream will be as continuous as possible.

6. The continuous and successful running depends on the uniform mixture. Be careful and not get it too thin.

7. At a shutdown of over twenty minutes flush out pipe by sending up 10 ft. of water in skip.

8. At the end of the day's run clean out mixer with two charges of water. Send same up through pipes.

9. All pipe joints should be oiled with thick grease, so as to prevent sticking.

With the wheelbarrow, cart, car or bucket method, the dumping of a large mass is apt to bridge over, or pocket, and wash itself when dumping, producing a variable mass, giving rise to laitance. There is absolutely no assurance of dense or solid formation inside of columns, girders, or even floor slabs.

Having demonstrated the above advantages of the Gravity System, it became necessary to evolve a general line of appliances, mechanically constructed, easily rigged, and absolutely "fool proof," which would handle concrete economically. Considerable experience and large amounts of money have been spent to determine the maximum and minimum angles of flow, the maximum and minimum reach of booms and free ends, the comparative relative value of differently shaped trough, and the advantages of the trough over the closed pipe. A typical installation is shown in Fig. 1 and the pouring of concrete in Fig. 2.

Concrete has been run successfully from 7 to 45 deg., dropped vertically 120 ft. into a hopper and redistributed. The best angle and the one now used by engineers is that of 18 deg. with the horizontal.

Different shapes of trough have been experimented with, from the flat, wide trough to the V shape trough, the V shaped trough and pipe from 4 to 10 ins. in diameter. The outcome has been that the 7 or 8 in. closed pipe is large enough for all capacity and light enough to be easily handled. The latest and really the only fool proof outlay is that of an 8-in. trough having open trough swivels and open trough free end. This allows the novice to do anything he wishes without injury to the plant.

The rate of flow of concrete was determined for practical reasons by Messrs. Uhden and Turner, Engineers of the Washington Water Power Company, Spokane, Washington, and it was found that through a round opening of 7 ins. emptying into an 8-in. trough, at an inclination of 18 deg. with the horizontal, mixed concrete of the above consistency ran at the rate of 132 cu. yds. per hour. The outcome of this investigation was the signing of a 100,000 yard job for a dam to be constructed by the Washington Water Power Company on the Spokane River. This will be the largest overflow dam in the world, being 200 ft. high and 400 ft. wide. It might be here mentioned that the Gravity System is being used on government work at Fort Mason, San Francisco.

The data in Fig. 3 and Table I has been deduced and formulated for the use of engineers in laying out the Gravity System.

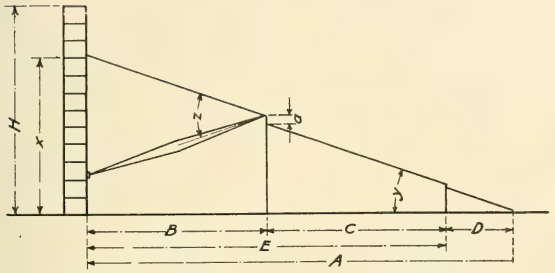

FIG. 3.—DIAGRAM OF GRAVITY SYSTEM APPARATUS.

A = Extreme or total length from tower to be reached.
B = Boom.
C = Free end of pipe, 30′–0″ may be 40′–0″.
D = Chute (10′–0″ to 20′–0″ long). D = Distance at corners of building.
$B = A - (D + C)$. $B = \frac{1}{2} A$ for reach of 60′–0″ and under.
$Y = 15°$. Tan $15° = .27$.
$Z = 18°$. Tan $18° = .32$.
a = 4′–0″ high for swivel.
$x = C \tan Y + B \tan Z + 4'–0''$.
$H = x + 16'–0''$.

TABLE I.—DIMENSIONS FOR PLANTS OF VARIOUS SIZES.

See Fig. 3. All dimensions in feet and inches.

A	$2A$	E	B	C	D	a	x	H
60–0	120–0	60–0	$\frac{1}{2} E$	$\frac{1}{2} E$	....	4–0	23–0	39–0
70–0	140–0	70–0	40–0	30–0	....	4–0	25–2	41–2
80–0	160–0	70–0	50–0	30–0	....	4–0	28–1	44–1
90–0	180–0	80–0	60–0	30–0	....	4–0	31–4	47–4
100–0	200–0	90–0	60–0	40–0	....	4–0	34–0	50–0
110–0	220–0	100–0	70–0	40–0	....	4–0	37–2	53–2
120–0	240–0	110–0	80–0	40–0	....	4–0	40–5	56–5
130–0	260–0	115–0	90–0	40–0	....	4–0	43–7	59–7
140–0	280–0	125–0	100–0	40–0	....	4–0	46–9	62–9
150–0	300–0	130–0	100–0	40–0	10–0	4–0	46–9	62–9
160–0	320–0	140–0	100–0	40–0	20–0	4–0	46–9	62–9

A short method of determining the necessary height of tower to deliver concrete for any known distance is to multiply the total distance to which the concrete is to be delivered from the tower by the tangent of 20 deg. This will give approximately the height of tower.

Referring to length of boom and free end, Table I, it will be noted that under the column marked A the distance of 160 ft. is the maximum radius at which it is practical to handle from one tower. This gives, as noted under column $2A$ a diameter of 320 ft., the boom in this case being 100 ft. in length. A boom of this size has considerable inertia, and does not operate as readily as one of about 60 ft. A diameter of 200 ft. is rarely used in laying

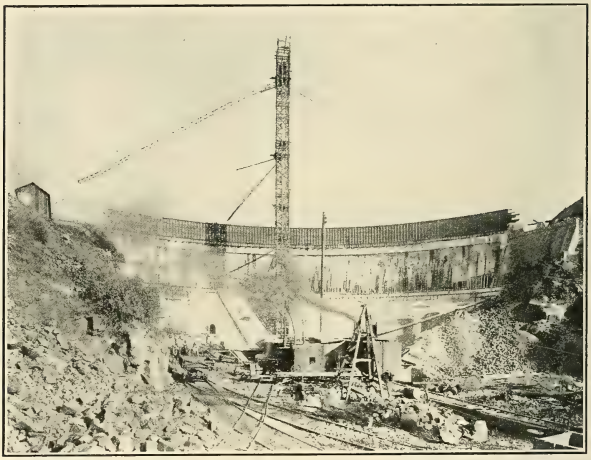

FIG. 4.—GRAVITY SYSTEM AS INSTALLED AT SWEETWATER DAM, SAN DIEGO, CAL.

out plans except in extreme cases where two or more towers cannot be installed.

The use of the Gravity System has demonstrated that economy is practical only when installation consists of boom connected to tower and rotating about same, supporting a pipe, swivel, and discharge or free end. This combination, as above stated, allows one or two men at the delivery end of chute or pipe to cover all points within a horizontal plane, limited only by the combined sweep of boom and free end. The installation further keeps all

apparatus pertaining to the distribution of the concrete above the operating floor, and on account of the large capacity of the system, it has developed ingenuity on the part of the engineers and contractors in constructing storage bins and loading devices to increase output of the mixer.

All details of construction have been gone into and are the outcome of actual experience on contract work.

The Gravity System is one of the most revolutionary methods in the handling of concrete as far as economy is concerned. The

FIG. 5.—POURING CONCRETE ON SWEETWATER DAM, ONE MAN HANDLING 25 CU. YDS. PER HOUR.

necessary men to operate consist of one tower man and two men at the delivery end of the pipe. Outside of this, the number of men used in tamping of concrete is reduced by one-half. The actual cost of the concrete varies considerably, but it has been found that the total labor cost of handling the concrete from the bins to its place in the wall varies from about 15 to 22 cts. per cu. yd. This includes a nominal royalty charge.

On the recent addition to the Sweetwater Dam, Fig. 4, J. D. Schuyler, engineer, and Arthur S. Bent, contractor, developed the fact that with the Gravity System it was much cheaper to put in concrete than place rock. Also in this case the contractors were

able to place about 15 per cent. rock as against 85 per cent. concrete. On dam construction this system is simply ideal, inasmuch as one man is able to simply carry the end of the pipe around, depositing the concrete among the boulders, Fig. 5, or wherever it is needed, doing away entirely with tamping and re-shoveling of the concrete.

It is in general not advisable for any contractor to use the Gravity System for the first time on a job of less than 2500 cu. yds. However, it has been successfully used on jobs as low as 250 cu. yds. In these cases the apparatus was on hand and the men were acquainted with its operation.

The cost of equipment of the Gravity System for large jobs is one-quarter that of cable tramway, and on buildings the cost is offset by the cost of scaffold and runways necessary for operations of carts or wheelbarrows.

In conclusion, this system has developed features which are truly revolutionary, inasmuch as it furnishes better concrete at a less cost than it has heretofore been able to obtain.

THE PREPARATION AND HANDLING OF CONCRETE.

By H. M. Cryder.*

In this paper "preparation" or the more common "making" of concrete will refer more particularly to the proper delivery of the materials to the mixer, rather than the proper proportioning and mixing of the same. The latter is now so nearly standardized, that its discussion would be uninteresting. "Handling" will cover the transporting and placing of the concrete. There will be illustrated and briefly described four or five methods.

The discussion is not in the least technical, and in mentioning costs it will be done only where the information is definite, gained from proper labor reports as well as actual contact with the work. It is realized how little it means to another, to show a photograph, give a short description of the concreting plant and state that concreting on that work cost only 37 cts. per cu. yd. There are almost as many methods of cost keeping as contracting concerns, and in fact, unless a contractor's work is confined almost wholly to one class of structures, say buildings, his cost system must be modified for nearly every large piece of work. However, some mention of costs in a comparative way will be made.

With a plant on any large job, designed consistently so that it has as much capacity to make, as it has to deposit concrete, the labor cost of concreting does not show much variance. Therefore, the problem becomes one of so designing the plant that the number of men required to operate, will produce this average labor cost per cu. yd. Then by estimating what the plant so designed will cost, very little is left for guess work on an item which is always large and vital, and which, quite often (for bidding) has to be much subdivided and then absorbed in small unit prices throughout different parts of the work.

Of course, a plant serving a large bridge, viaduct, or continuous wall work, must have a capacity of from 20 to 35 cu. yds. per hour, so that the usual short delays of a half hour or less or

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an extra man's time, do not materially affect the concreting cost, say more than 1 ct. per cu. yd. Therefore, as it is easy to increase the handling or depositing capacity by adding more cars or more chutes, the problem usually is how to handle the materials to the mixer, so as to absolutely maintain the figured or theoretical capacity of the whole plant.

FIG. 1.—PLANT AT COLORADO RIVER BRIDGE, AUSTIN, TEXAS.

Without dwelling on the faults or unreliability of wheel-barrows, bucket conveyors or apparatus which hoist wagons or small cars, experience gained from the operation of three comparatively large plants shows the canvas conveyor belt to be the best and most flexible method for bringing materials to the mixer cheaply, and as needed. It must be understood that this applies to bridge work, not buildings, although it is very adaptable to large buildings

where storage room and railroad tracks are available. Such a conveyor is practically fool proof, and its simple adjustments, together with the greater advantages of being able to increase its speed (and consequent capacity) in only a few minutes' time, make it almost ideal.

A view of the plant on the Colorado River Bridge, Austin,

FIG. 2.—PLANT AT KINGS HIGHWAY VIADUCT, ST. LOUIS, MO.

Texas, with the belt conveyor in action, is shown in Fig. 1. This conveyor consists of a main belt, about 400 ft. long, in a fixed position, and a branch belt, about 250 ft. long, pivoted on to the main belt, the branch belt swinging through a complete circle, in order to reach the necessary amount of material. It carried gravel, sand and bolders which were washed, screened, crushed and deposited in their proper bins, above the mixer. Eighteen men,

at $1.50 per day, loaded on this belt the raw materials for 200 cu. yds. of concrete per day. The cement was carried to the mixer from the shed shown to the right by another belt about 100 ft. long. The concrete structure was built on the center line of the old bridge, the steel spans of same having been moved 50 ft. downstream to temporary pile piers.

There was nothing unusual in connection with the method of handling the concrete on this work, a double track tramway,

FIG. 3.—CONCRETE HANDLING PLANT AT KINGS HIGHWAY VIADUCT, ST. LOUIS, MO.

24-in. gauge above the arches, with rotary dumping cars, having been used. The concrete was transported a maximum distance of about 800 ft. from the plant, the tracks having a fall of 7 ins. in 100 ft., which is sufficient, and has been used again, on a structure 1,900 ft. long, with the plant in the center.

Reverting to the conveyor and the making of the concrete, Fig. 2 presents a view of the belt layout on the Kings highway viaduct in St. Louis. This picture is taken from the roof of the elevated bins over the mixer, into which the belt discharges, and is looking

down the belt to the receiving yard where the material is handled from cars. Sand, crushed stone and gravel are stored separately under the elevated track on which the unloading crane is standing, and are fed (one at a time) onto the belt running in the tunnel beneath the trestle. The man at the top in charge of the bins, signals the belt feeder in the tunnel when to begin or stop

FIG. 4.—BRANCH TRAMWAY OVER PIER WITH CHUTES.

loading sand, gravel or stone. This is done by flickering the lights in the tunnel (so many times for each need) by a knife-switch in the circuit at the bins. The crane moves up and down the elevated track, supplying the materials where the hoppers have become empty. The great advantage of this layout is that materials can be stored ahead ready for immediate use, giving protection against poor delivery or bad weather. The bins above the mixer hold only

about 20 batches, enough to supply gravel while sand is coming up on the belt, or vice versa.

On the handling side of this plant, Fig 3 shows the mixing plant and the triple track tramway, running the length of the viaduct, 1,900 ft. This tramway clears the Missouri Pacific and Frisco tracks, seen in Fig. 2, and as the reinforcement, forms and

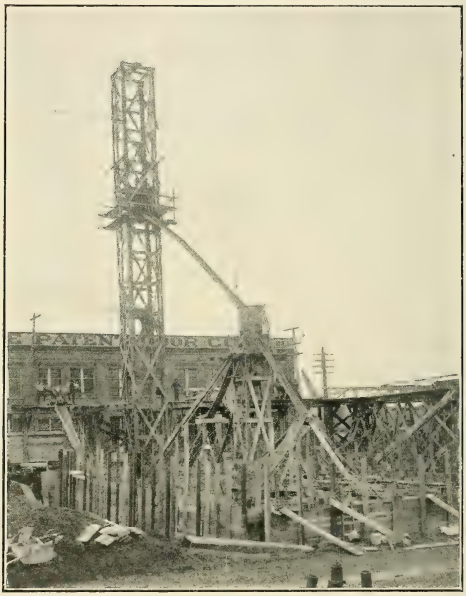

FIG. 5.—TOWER WITH CHUTES LEADING OVER PIER.

centering will be handled by a light locomotive crane traveling on the tramway, the danger of accident, on account of railroads, is reduced to practically the minimum.

As illustrations of the handling of concrete on this work, Figs. 4 and 5 show the methods at two of the large piers, the latter almost identical in design and not demanding different methods of handling, the contractors, however, tried the two methods to obtain

a cost comparison. In the first, called Pier 6, Fig. 4, a branch tramway was built over the center line of the pier, the legs of the bents being cut off, or sections removed, as the concrete was placed, until the pier was completed to the spring line of the arches—on account of the elevation of the permanent or main tramway, this layout could not serve further, and must be replaced by some means for elevating the concrete to the arches and floor system. This may be done by a derrick or locomotive crane, instead of a tower and chutes, as seen in Fig. 5 showing Pier 9. In this case, the tower has sufficient height to serve the arch rings and floor system, at a single setting, and while it requires power and a hoist from the beginning of its use, it may prove to be the more economical of the two schemes. The collector, as shown in the center, is set at the proper elevation for serving the whole pier, the branch chutes, only, being changed as the pier walls go up. For the arch rings and floor system, the main chute and collector will be raised. The hopper cars, coming from the central plant, are run onto the cage of the elevator tower, elevated and dumped direct into the steel chute, which leads to the collector, holding about a yard and a half. This collector has a convex bottom (not concave) with five gate openings, by which the branch chutes connect and are fed. With this arrangement, by having from two to four branch chutes, concrete can be delivered to as many different points, or while one chute is in use, another can be set, so that there is absolutely no delay from moving or re-adjusting chutes. Further there is usually no delay in moving cars, as the collector holds about two carloads, thus bridging over any small delay in depositing.

Attention is called to the type of cars employed; fixed hoppers, with undercut radial bottom gates. The ease with which the outflow of the concrete can be controlled, and their consequent speed, are the advantages making them the best car.

Diverting from the swift, noisy and mechanical making of concrete in this more progressive country, to the quieter, slower methods of Old Mexico, Fig. 6 shows an all concrete building in Mexico City. The outside walls, window sills and lintels, main entrance arch, projecting quoins and all details seen, were poured monolithically, which statement is meant to emphasize the second division of our subject—handling. The materials to the mixer,

and the concrete from the elevator were handled in wheelbarrows, but the concrete was mixed and placed, not necessarily so carefully, but so properly, that there was produced an outside finish which after rubbing showed no flaws or discolorations worth consideration —and there is no plastered surface. There are really only two main points to the term "proper handling." First, to get a wet mix, without much surplus water (an absolutely vital point to the contractor and not the engineer), since a better, more uniform

FIG. 6.—REINFORCED CONCRETE BUILDING IN MEXICO CITY, MEXICO.

grade of concrete is assured, and second, to place it in truly level courses, with numerous bulkheads, to predetermined and marked heights; such as the tops of windows, the bottom of a belt course or cornice, or the sunken joints between quoins.

Such a building would be difficult of construction in most parts of this country, because of the contractor's inability to absolutely control, from two or three standpoints, the method of producing the desired results.

This building has a monolithic raft foundation, of reinforced

concrete slabs and beams, which type distributes the load evenly on the unstable soil of the valley of Mexico assuring even settlement, if any, instead of the very common freakish settlement of nearly all of the buildings in Mexico City. However, this is not the occasion for further discussion of the design, and the soil.

The labor cost of concreting the raft foundation, columns, girders and slabs of this building, was about 45 cts. per cu. yd.—with wheelbarrows as previously stated, and this cost includes, bringing to the mixer, depositing, facing and the time of the fore-

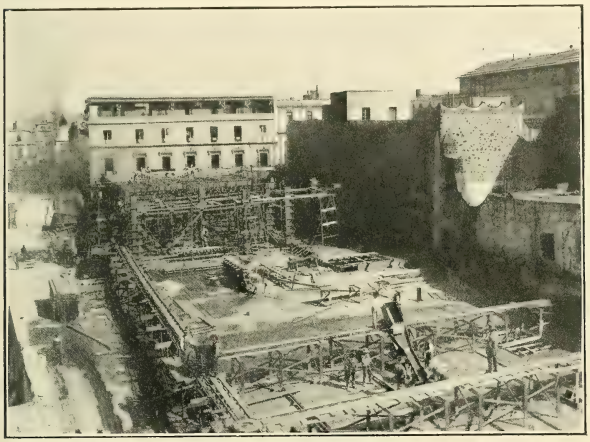

FIG. 7.—CONCRETING PLANT, HOTEL GANTE, MEXICO CITY, MEXICO.

man. The Mexican common labor cost kills the economy of most any mechanical plant that would be economical and necessary in our country.

Not being satisfied, however, with the appearance of the wheelbarrow method, regardless of its satisfactory cost results, a track and rotary dumping car layout, Fig. 7, was used on the next contract in Mexico City—The Hotel Gante. With this layout, having the same superintendent, mixer, labor wages and cost reports, as on the building shown in Fig. 6, the cost of making and placing was reduced 11 cts. per cu. yd. The cars and track cost 7 cts. per cu.

yd. more than the wheelbarrows, considering both as capitalized investments, so with the improved plant a net gain of 3 or 4 cts. per cu. yd. was produced. It is quite certain, from an estimated layout, that the chutes would not have produced any more saving,

FIG. 8.—ROTARY DUMPING CAR AND PLANT, WABASH RIVER BRIDGE, IND.

considering, of course, that the chutes themselves and their rigging, would have cost more there, than here. This building also has a raft foundation, designed and guaranteed against failure, by the company building it, for a Mexican architect.

Next is the consideration of the construction of five bridge

piers for the Wabash Railroad over the Wabash River in Indiana; the work being of a different type than those so far considered, and the method used quite different from what would likely have been used ten years ago—when derricks and buckets would have made it a hard and longer job. A rotary dumping car, ready to be loaded with the dry materials for the mixer, at an elevation 15 ft. below the level of the railroad tracks, is shown in Fig. 8. Material from the stock piles is shoveled into small portable hopper bins (one showing above the car body) straddling the 24-in. gauge

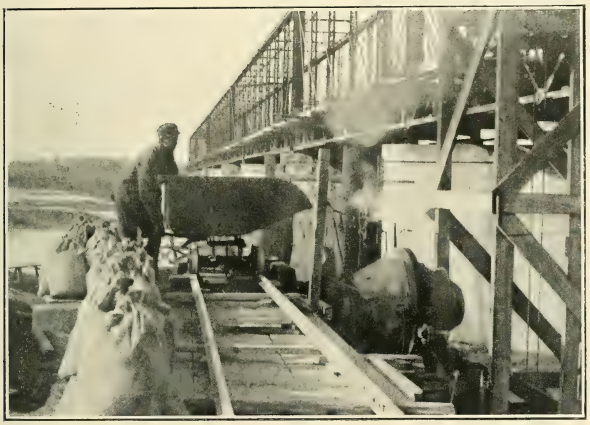

FIG. 9.—ROTARY CAR LOADED WITH MATERIALS READY FOR DUMPING INTO MIXER.

track, carrying the dump car. The empty car was run under the bin farthest from the mixer, and the proper amount of gravel (gauged in the bin) dropped into it, through a slide bottom gate. The car is then moved to the next bin for sand—then given the cement last before reaching the mixer, where the whole batch was dumped at once, as shown in Fig. 9. This scheme worked perfectly, eliminating any confusion at the mixer, as can be readily understood. In cost it was quite economical over wheelbarrows. From the mixer the usual elevator and skip were used, Fig. 10, and

the handling was done in the same rotary dump cars. The track for the cars was laid on wooden cantilevers, Fig. 11, placed over the bottom cord and under the railroad track ties—a cheap and safe arrangement.

One man handled the loaded cars on the short haul, but two men were put on the long haul, which was about 700 ft. The cost of taking materials from the stock piles, mixing, transporting and

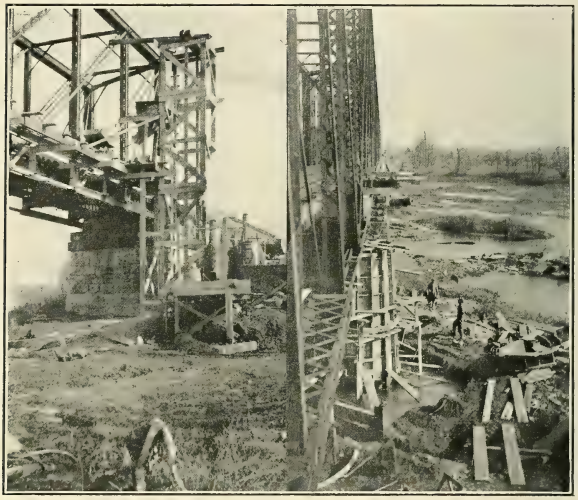

FIG. 10.—PLANT AT WABASH RIVER BRIDGE, IND.

FIG. 11.—TRAMWAY ON WOODEN CANTILEVERS.

placing in the forms, was 70 cts. per cu. yd. on the long haul, at the last pier.

It is such situations as the one last described that are considered more ideal for a paper of this kind than would be the detailed description of the plant, handling and cost deductions of some large work, the parallel of which might happen only once or never in the experience of the average engineer or contractor.

In what has been said it is not meant to convey the idea that

all or any of the descriptions represent the latest or so-called most improved methods for making and handling concrete—they are, however, descriptions of plants which were actually built, which worked satisfactorily and were sufficiently modern to allow, primarily, of obtaining the contracts, in competition, and which, afterward, proved economical when figured against the original estimates. The author could not have described or recommended other attractive looking plants, with which he was less intimate, with any degree of faith, and so has remained within the sphere of actual experience.

The subject of chutes, in connection with handling, has not been touched as another paper* deals with this subject exclusively. The hopeful feature about spouting is, that the different systems are rapidly being changed and improved all over the country by reputable contractors and progressive engineers, whereas a year or two ago, this method was being abused to such an extent that a great deal of objection and condemnation arose.

There is, in concluding, an imminent demand for a method of conveying concrete on important structures, such as long viaducts, walls or long buildings, which will reduce the cost of trestle work necessary for tracks operating cars, or of extravagant spouting layouts, and such schemes as the horizontal or flat conveyor, or even a system of pipe lines with the vacuum principle applied must be considered, studied and used,—for such improvements will bring, with their perfection, uniform and perfected concrete, and hasten the universal use of concrete on any structure which should be permanent or fireproof, because the other need, before concrete really comes into its own,—a more uniform and much lower cement price—is believed to be already in sight. Those of us, called the Cement Users, should not be found inefficient or undeveloped when, in a short time, appearance and quality alone will remain as the objections against concrete, the universal material of construction.

* "The Spouting of Concrete," by Fred E. Engstrum, p. 522.—Ed.

MOVING FORMS FOR REINFORCED CONCRETE STORAGE BINS.

By James MacDonald.*

The use of Portland cement will reach its highest development when methods of molding it have reached the maximum efficiency at the minimum cost. The expense of the form or mold, particularly for superstructure work, is an important item in the total cost of any structure built in concrete. In most instances the salvage from a set of forms having been once used, is small and can seldom be utilized the second time. We are now in the transition period of building construction, with architects and engineers adapting themselves and their plans to the employment of cement construction in place of wood and masonry. The tendency during this period is and will be to adhere to the outlines and designs in structures which were permissible and desirable with old materials, but which can only be imitated at much expense in the forms by using concrete. For instance, the outlines of a foundation wall in rubble stone or brick may be battered or curved without adding materially to the cost of the work. In concrete, the complicated lines and angle cuts in the form will consume much more lumber and labor in their construction than the same wall would require if built with vertical parallel lines, and in nearly all cases the conditions for concrete construction will permit the design to be made with a view to simplicity in the form. It may often pay to sacrifice a volume of concrete, by using more than may be actually necessary, if by so doing simplicity may be attained in the mold construction.

Reinforced concrete is now very generally used in the construction of storage bins for coal, grain, sand, crushed stone, and similar loose materials. A scientifically correct calculation will show that the walls may be much lighter at the top than at the bottom in any of these storage bins. This would necessitate "battered" or "offset" construction in order to follow the theoreti-

* President, MacDonald Engineering Company, Chicago, Ill.

cally correct lines. If, however, the walls are built with their maximum thickness at the bottom, and this thickness maintained at the top, in order to get parallel vertical lines in the wall, the structure may be built at the minimum cost for the forms by using a moving form. Vertical walls in any shape, cylindrical, square, multilateral or curved, may be cheaply built by using a moving form, so long as the idea of uniform thickness and vertical parallel lines is carried out in the design.

The moving form consists essentially of a mold in which the finished wall shapes are accurately outlined in the enclosed walls of the form. Wood or metal may be used in their construction, but for most cases wood is for many reasons the best. The depth of the mold need not exceed 4 ft. for any ordinary construction. The ceiling boards may be used from 7/8 in. stock, 4 ins. wide, dressed and matched, and placed vertically in the mold wall, so that the grain and joints will be parallel with the vertical movement of the form. The skeleton frame on which the mold is built should be rigid, and able to stand heavy strains. The wall space in the mold where the concrete is to be poured must be free from all obstruction. Consequently the spacing apart of the adjacent surfaces, which represent the thickness of the wall, must be controlled by a sufficient number of arch yokes rigidly secured to both sides of the mold so as to maintain the wall space.

The jacking mechanism by which the mold is moved upwards is usually attached to and forms a part of the yoke. The forms should be built to cover the entire circumference of the structure to be erected, the only openings being the mold openings where the concrete is to be poured, all other areas being solidly floored over. This gives an unbroken scaffold area for men and materials during construction.

The jack used for raising is a device which gets its lifting support from a vertical solid rod, about 1¼ ins. diam., or 1½ in. pipe, which is buried vertically in the walls, the lower end resting on the foundation. The jack forms the connecting medium between the supporting rods—which are always left in the structure—and the mold. The mechanism of the jack slowly climbs the supporting rod, at the same time carrying the mold upwards with it. This upward movement strips the mold from the concrete and

prevents adhesion between the wall and the mold surfaces. In starting the operation, the concrete is poured in the mold to an average depth of about 1 ft. It is then allowed to get its initial set, which requires about 4 hours. The mold is then gently and uniformly started upwards, lifting about ½ in. at a time at each jack. Another foot of concrete is then poured and the mold moved. This is continued until the mold is about filled with concrete. At this point the concrete, first deposited, should be about 10 hrs. old, and will be found strong enough to be self-supporting in temperatures above 50 deg. From this time on the movement

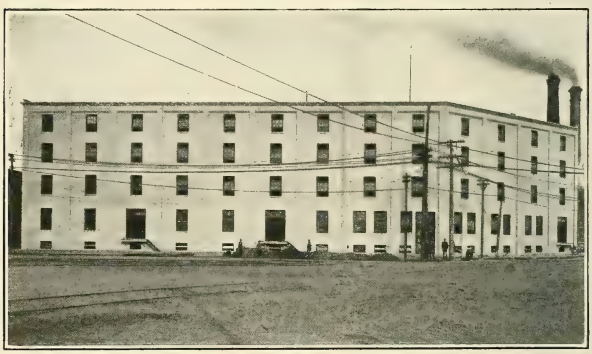

FIG. 1.—REINFORCED CONCRETE WAREHOUSE, EVANSVILLE, IND.

should not cease night or day until the work is completed. The reinforcement is placed in its proper position as the mold moves upward. The mold should be kept continually filled, and at no time during the progress of the work should the concrete in the top of the mold be allowed to attain its initial set.

In practice it is found that the mold may be moved up, and the wall built at the rate of from 6 to 8 ft. in 24 hrs., depending on the temperature of the air. The rate of progress will be governed by the setting of the concrete. It will be noted that the green walls when being formed and exposed, have nothing to carry but their own weight. The supporting medium for the form itself, the men and materials used in the construction, all of which are

carried on the form, is transferred through the jacks on the vertical supporting rods or pipe to the foundation. As the walls grow in height the concrete sets firmly around the supporting rods, and gives them the lateral support which is needed as the distance increases from the foundation.

There are several devices used for the jack mechanism, most of which are patented. One style of jack has a threaded rod, with a course pitch, engaging a nut which may be clamped to the supporting bar, the screw itself being attached to the mold through the medium of the yoke frame. The screw has a limited working

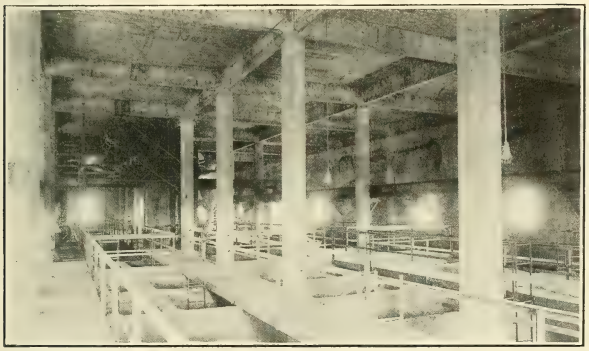

FIG. 2.—INTERIOR OF MILL PLANT AND GRAIN ELEVATOR, MEMPHIS, TENN.

range of about 24 ins. When this limit is reached the nut is released and clamped to a higher point on the supporting bar, and correspondingly to the starting point on the threaded rod. The jack is then ready for lifting the load another 24 ins. Other jack devices have friction attachments connected to the supporting bars. The mold is lifted by a lever which is pivoted to the grip and connected to the yoke by means of suitable link connections. The green concrete cannot in any case be employed as a support or fulcrum for lifting the mold. Some of the jack devices employed are made so that the lifting leverage is pivoted at a point outside of the vertical center of the rod support. This gives an eccentric load on the rod, and is apt to produce a springing

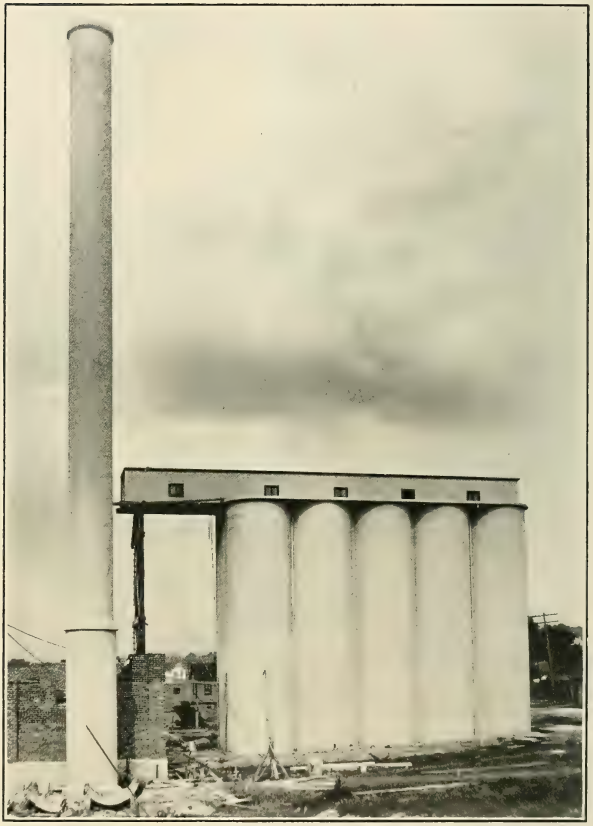

FIG. 3.—GRAIN ELEVATOR AND SMOKESTACK, JOHNSON CITY, TENN.

movement laterally, which might give trouble by shattering the concrete.

The raising of the form should be steady and uniform, keeping all parts level and without distortion of the concrete. By making the operation continuous the concrete is left monolithic and water tight. If the forms are allowed to stand over night the concrete is very apt to grip the mold surface, and often cannot be freed without shattering or damaging the concrete. This sometimes makes it necessary to dig the entire body of concrete out of the

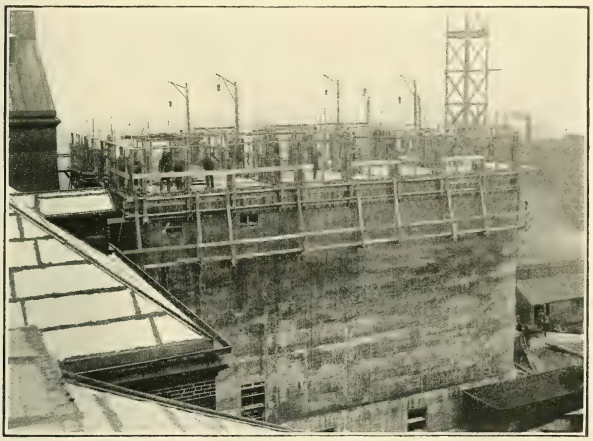

FIG. 4.—GRAIN ELEVATOR UNDER CONSTRUCTION, INDIANAPOLIS, IND.

mold, and start fresh. When the concrete in the partially filled mold is allowed to set, it is very difficult to avoid a bad joint when the next concrete is poured. The cement splashings around the mold dry up and crumble into scuffy sweepings, which lodge between the two concrete surfaces, and keep them apart like sawdust. The wall finished in this way is apt to leak during a driving storm, and the appearance of the joint is clearly outlined in the concrete surfaces.

The moving form gives the builder an opportunity to give the concrete an excellent finish. The surfaces may be troweled and

washed with cement, while the concrete is green and plastic. Defective spots may be pointed up, and the whole surface brought to a uniform finish and even color before getting its final set. It is a great convenience if the form is prepared so that it can be easily and conveniently stripped from the building at the finish. A little forethought on this point at the commencement may save much time and expense at the end.

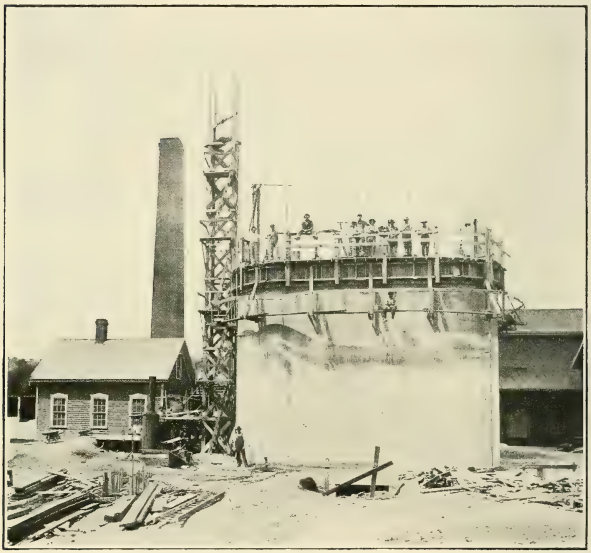

FIG. 5.—GRAIN ELEVATOR UNDER CONSTRUCTION, LOUDENVILLE, OHIO.

In 1906 a five-story warehouse, Fig. 1, 85 ft. wide and 190 ft. long was erected at Evansville, Ind., with moving forms which stood 4 ft. high on the foundation. The outer walls and interior columns were both poured in the form as it moved upward. At the different floor levels the girders were laid in and supported independently after being poured. The form was moved up into the next story, all windows and door openings were placed in their

proper positions while the form was slipping past them, and each story with its columns and girders were molded this way until the roof was reached. The roof slab was then spread over the entire area of the form, which was left in place while the roof was set up. At this stage the building had four walls and the roof with all the columns and girders in position. By the time the floors were put in, the columns and girders had attained sufficient strength to not only stand unsupported, but to carry the hanging scaffold for the floor slabs, all of which were poured independently after the building was enclosed. Fig. 2 is an interior view of a mill plant and grain elevator at Memphis, Tenn., constructed with moving forms.

The moving form has a wide range of usefulness in concrete construction, and in many cases may be used to displace large amounts of labor and material now wasted with fixed molds. In every case where it has been employed the results have been satisfactory, the concrete is uniform in appearance, and the cost much less than can be obtained with the fixed form. Grain storage buildings now built, without the moving form, would be impracticable both in construction and in price. Concrete has in five years practically displaced the use of tile and brick for this purpose. Fig. 3 illustrates a grain elevator and smokestack at Johnson City, Tenn. Figs. 4 and 5 show grain elevators under construction with moving forms.

The Washington Monument can be duplicated in reinforced concrete above the foundation in 100 days by using the moving form. The things that have been accomplished by this method of forming have proved its usefulness and economy, and the possibility of extending its field of operation is unlimited.

REPORT OF COMMITTEE ON TREATMENT OF CONCRETE SURFACES.

The Committee submits herewith a further progress report not with the idea that it is perfect or complete but with the strong desire that it form a basis for discussion and that from the discussion a more complete and concise statement of the subject can be prepared as well as specifications which will enable anyone fairly experienced in the art to do good work.

Plan and Scope.

A synopsis of what the Committee proposes to do is given below. It is not assumed that this synopsis is complete, and any subjects which should properly be treated by this Committee and which have been omitted, will be added.

I. Effect of Material and Workmanship.

(*a*) Cement.
(*b*) Fine aggregate.
(*c*) Coarse aggregate.
(*d*) Foreign matter, active or inert.
(*e*) Water.
(*f*) Pigments.
(*g*) Molds.
(*h*) Mixing and placing.
(*i*) Treatment while hardening.

II. Removal of Surface.

(*a*) Unskilled labor. Picking, scrubbing, rubbing.
(*b*) Skilled labor. Crandall, bush hammer, tooth tool, chisel, set, lathe.
(*c*) Sand blast.
(*d*) Chemical treatment (kind, strength, method).
(*e*) Age when cut or treated.
(*f*) Tools. Kind, quality, temper.
(*g*) Effect of cutting on permeability.
(*h*) Effect of cutting on durability.

(*i*) Effect of cutting on collection of dirt.
(*j*) Advantages and disadvantages of certain treatments.
(*k*) Photographs.

III. Coatings.

(*a*) Texture and condition of masonry surface to be covered. (Masonry includes brick, terra cotta, stone, concrete, mortar and plaster.)
(*b*) Metal lath. Kinds, method of erection.
(*c*) Preparation of base to obtain a bond.
(*d*) Material. Cement, sand, lime, pigments.
(*e*) Mixing.
(*f*) Placing. Number of coats and treatment of each, workmanship, texture.
(*g*) Joints.
(*h*) Washes.
(*i*) Paint.
(*j*) Enamel.
(*k*) Durability.
(*l*) Photographs.

IV. Ornamental Work. Possibilities and Limitations.

(*a*) Cast concrete.
(*b*) Cast mortar.
(*c*) Plastered mortar.
(*d*) Kinds of molds. Wood, iron, plaster, glue, sand (plain or treated).

V. Waterproofing.

(*a*) Necessity.
(*b*) Mixed in concrete, either as a powder or a liquid.
(*c*) Method of incorporation.
(*d*) Surface treatment without changing color.
(*e*) Surface treatment changing color.
(*f*) Mortar.

VI. Limitations, Defects, Blemishes of Various Kinds and Remedies.

(*a*) Range of colors.
(*b*) Variation of color.
(*c*) Craze cracks, checking.
(*d*) Expansion and contraction cracks.

(*e*) Irregular size of castings.
(*f*) Mortar and stone at horizontal joint between days' work in concrete wall.
(*g*) Patches.
(*h*) Efflorescence. Cause and remedy.
(*i*) Porosity. Cause and remedy.
(*j*) Frozen surfaces.
(*k*) Dusting.

VII. Costs.

Exact and relative cost of each method, and manner of estimating cost of materials and labor.

VIII. Specifications for the various treatments described under I, II, III, IV and V.

I. EFFECT OF MATERIAL AND WORKMANSHIP.

(*a*) *Cement.*—The color of the surface is due to the cement and aggregates used, and in the case of a scrubbed surface it is largely due to the aggregate. Although there is some difference in the shade of gray of standard Portland cements, it is so slight that with very few exceptions no thought need be given to the color of the cement. There are white cements on the market which afford very pleasing treatment of the finished surface. It is doubtful whether the seasoning of cement has any influence on hair cracking, crazing, surface shrinkage or efflorescence on the surface of concrete. Cement which has become caked in the bag must be broken up and sifted before using.

(*b*) *Fine Aggregate.*—The fine aggregate is in general suitable material less than ¼ in. in size.* If the surface is tooled or scrubbed particles of aggregate will be exposed to view and their color will have an important influence upon that of the surface. Their size will also have an influence upon the texture.

A sand with large round grains such as is found in places on the sea coast, the grains being practically of one size, cannot be used in as lean a mixture as sand of variable size and angular grains, because the round grains will rub from the surface with

*For further definition see *Proceedings*, Vol. V, p. 456, or Vol. VI, p. 351.—ED.

considerable readiness if the mixture is 1 to 2 or leaner, and as the grains in the first named sand are large they have more influence upon the color than sand of variable sizes. The coarse Plum Island sand used in Boston and vicinity has grains of uniform size and is of a yellow color, and work done with it has the appearance of being iron stained. A light gray or white quartz sand will very materially lighten up the color of the mortar and give the appearance of the use of some pigment. Stone dust makes an excellent sand and when of a grayish color can scarcely be distinguished from that of the cement, thus giving a uniform appearance. Quite a variety of colors and textures can be obtained by care in selecting an aggregate, either natural sand or stone dust, of variable size grains, ranging from that which will pass a 30 mesh sieve to that which would just barely be rejected upon a ¼ in. sieve.

(*c*) *Coarse Aggregate.*—The coarse aggregate is in general suitable material over ¼ in. in size.* The coarse aggregate, still more than the fine aggregate, can be used to vary the appearance of the finished surface on account of its larger size and ease with which it can be cleaned of cement when the latter is still green. It is possible to get a very wide range of colors in stone and either one color can be used or two or more can be mixed together with very pleasing effects. The size should range from rejection of the ¼-in. sieve to that which just passes a 2-in. ring. Contrasting the angularity of broken stone with the round and smoother surfaces of gravel, gives effects which are still more varied than with quarry stone, and are just as durable so far as strength and watertightness are concerned.

(*d*) *Foreign Matter.*—It is also possible to get some pleasing effects by the introduction of foreign matter such as broken brick, glass, fragments of metal like copper, brass. Even nails are sometimes used, but the rust and stain from the iron is unsightly. Ores, such as green copper ore, have been used for producing certain effects. Oil, glue or lime have also been introduced at times for certain purposes.

In one instance the following mixture was used: To every

* For further definition see *Proceedings*, Vol. V, p. 456, or Vol. VI, p. 351.—Ed.

barrel of cement were added 4 lbs. of salt and 20 lbs. of quicklime, slaked for one week and passed through a fine screen, 4 lbs. of lampblack and 1 lb. of linseed oil. In preparing this the lime was first slaked and while yet hot the salt in solution together with the lampblack and oil, were added and all stirred thoroughly together, the mixture then being passed through a fine screen into vats and allowed to stand for one week or longer before using. The proportion of aggregates to the cement was 6 to 1, the stone being a mixture of dark blue trap and white marble. After exposure to the weather for 12 years, the surface has proven durable and free from defects. The salt was added to hold moisture so that the cement would set slowly. The lime and oil were added to increase the density of the mass, and the lampblack to obtain a darker color.

(*e*) *Water.*—Water has its influence on the appearance of the surface in several ways. The amount of water has an influence on the density. When the mixture is quite dry it is impossible to compact materials well, and the resulting mortar or concrete is porous. If a sufficient amount of water is used to produce a plastic mix the appearance is dense and uniform, while a large excess of water brings the fine particles to the surface and produces a little different texture from plastic. Moreover, the quantity of water affects somewhat the time of set of the cement and thereby to some extent the color.

(*f*) *Pigments.*—Coloring matter of various kinds can be added to the mixture. Those of a vegetable origin are not permanent because they are likely to be acted upon chemically by the ingredients of the cement. Only mineral colors should be used to obtain tinted mortars for concrete or stucco surfaces, as follows:

Gray	Lampblack, manganese dioxide.
Slate	Lampblack, manganese dioxide.
Red	Red iron oxide.
Bright red	English red oxide.
Brown	Brown roasted iron oxide, brown ochre
Yellow	Ochre.
Blue	Ultramarine.
Green	Chromium oxide, ultramarine green.
Violet	Violet oxide of iron.

The amount which can be safely used is small, owing to the danger of impairing the strength of the resulting mortar. A quantity equal to 5 per cent. of the cement by weight should be the limit, beyond which the impairment of strength is too great to justify a larger amount. Even this amount of certain colors which differ little from that of the cement, like yellows and reds, do not produce a marked change from the color of neat cement. Lampblack is the principal ingredient used for darkening and a moderate amount produces a strong contrast on account of the great difference in color and the fineness of its particles. For lightening the color somewhat, lime is the best material to use. Coloring matters can be used in either a dry form or as a paste. It will generally be found most convenient for mixing to use the dry form, thoroughly mixing it with the dry materials before the addition of water.

(*g*) *Molds.*—Molds also have an influence upon the appearance to some extent. With rigid materials like metal or wood there is a tendency for the coarsest particles to come to the surface, and unless this is overcome by spading or proper manipulation the surface is likely to be rough and porous, and if the stone is pushed back too far a mortar face is obtained. With rigid molds which are non-absorbent, the surface is likely to be of a watery appearance and on hardening small holes appear where drops of water evaporated. Porous molds, such as sand or various combinations of sand and other substances, treated or untreated with chemicals, all produce a similar effect, namely a dense cement surface, as the cement is brought to the surface by the excess water which is absorbed by the molds. However, this type of mold does not remove cement from the mortar and its surface of contact is not bonded to the casting by the excess of cement which comes to the surface. This excess of cement on the surface is liable to cause hair or map cracks and in large specimens is more subject to shrinkage than with molds of other material. If the mold is not filled by continuous pouring an unsightly line, impossible to conceal later, is left where work stopped.

(*h*) *Mixing and Placing.*—On account of map cracking and shrinking it is not wise to mix mortar richer than one (1) part cement to two (2) parts sand, and for freedom from these two

defects it is frequently better to mix it as lean as one to four (1:4). Stone in proper proportions can be added to the mortar to get the desired textures. Thorough uniform mixing and quick placing are desirable features. The aggregates should be graded with the utmost care so as to obtain a maximum density. A concrete whose surface has to be removed should be so made that it will have an absorption of not more than 5 per cent. by weight. These subjects have been so well covered in the past contributions* to this Association that detailed treatment is here unnecessary.

(*i*) *Treatment While Hardening.*—The appearance can be modified somewhat by the treatment of the surface while hardening. If it is allowed to dry rapidly it will be of a lighter color than if kept damp and cool and allowed to dry slowly. Precautions should be taken to prevent rapid drying. Keep the surface wet for at least 4 days. Rapid drying makes the surface friable, craze and dust.

II. REMOVAL OF SURFACE.

(*a*) *Unskilled Labor. Picking, Scrubbing, Rubbing.*—A variety of treatment can be given to the surface by the use of unskilled labor.

Picking.—Taking such tools as a hand pick with a single point or a pick with a face of 2 ins. with a series of points upon it, the surface may be scaled from the concrete, entirely removing any marks of the mold or any stains from a coating placed upon the surface of the mold to prevent adhesion of the mortar, exposing the sand grains and the stone of which the concrete is composed. The cost varies considerably with the age of the cement when the picking is done. When done at the age of 3 or 4 days a laborer can cover four to five times the area he can when the cement is 2 or more weeks old. The texture, however, differs. As a larger amount of material is scaled off when the concrete is quite green and scaled off to varying depths and as some particles of stone are chipped out of the surface, the texture is much coarser than when treated after the cement has become hard. Under the latter conditions the surface has a fine texture and is quite uniform. A little

*Selecting the Proportions for Concrete, William B. Fuller. *Proceedings*, Vol. III, p. 95. Proportioning and Mixing Cement Mortars and Concrete, L. C. Wason. *Proceedings*, Vol. IV, p. 129.

variety can be obtained by the method in which the blow in picking is applied. By striking a perpendicular blow no resulting mark is left on the surface by the tool, whereas with a glancing blow lines are left, and these can be all parallel to one another or at various angles, and thus a little variety to the texture can be given.

Scrubbing.—Another way of treating with unskilled labor is by scrubbing with water and stiff brushes, either bristle or wire, while the mortar is still quite green. This removes the excess cement from the surface, obliterates the marks of the molds, and washes considerable of the cement from the surface of the aggregates, so that they appear cleaner, sometimes as clean as freshly-broken stone, and thereby obtaining a bright and pleasing effect. It is common also, in this method, to use an acid which will attack green cement rapidly and eat it from the surface of the aggregates. This type of treatment is used usually during the first 48 hours after the cement has been cast. After this age the cement resists the scrubbing and acid process to such an extent that it becomes expensive, and the best results are not obtained.

Nitric, muriatic and sulphuric acids in various strengths up to the undiluted have been tried, and experience is that these agents are impracticable for the removal of vertical surfaces of *good* concrete thoroughly hardened, the action being so slow and attended with so much difficulty that hammering of one sort or another is cheaper.

Great care must be taken in successive casting to prevent marks showing (after scrubbing) where the new and old work join. The practical operation is to finish casting on some line in the building, although in certain architectural treatments this stratification is not objectionable. When it is proposed to scrub the surface of the concrete before it is more than 24 hours old, it is necessary to design the forms specially for prompt removal without damage to the rest of the structure. This will usually save lumber, but designing for quick removal will usually add to the labor an amount sufficient to offset this. Moreover, arranging to make possible the removal of forms at certain points of the work does not necessarily permit maximum efficiency of the mixing plant, as the quantity of work may not be such as to run the plant at full capacity for a day.

Rubbing.—Another way of treating flat surfaces is by rubbing with pieces of limestone, water and sand, cutting away the mortar and surface of the stone, so as to expose the full cross section of the aggregate. This is done when the mortar is 3 or 4 days old, and only with the softer stones, such as marbles. This type of finish is usually known as terrazzo.

A pleasing treatment of all surfaces, giving an even light color which experience has proved to be durable, is as follows: When the cement is green rub off first with No. 8 carborundum brick, wet surface, and fill voids with a mortar composed of one (1) part cement, two (2) parts fine sand, and float the surface, leaving the least amount of material possible on the wall. Let it stand for any length of time. As a final cleaning down of the wall surface, wet thoroughly, brush on a small amount of mortar, and immediately rub off with a No. 30 carborundum brick.

The following is the method used in treating the concrete telegraph poles along the Pennsylvania Railroad track across the Hackensack Meadows. The surface of the concrete is thoroughly wet with clean water, then rubbed vigorously with a cement brick made of one (1) part Portland cement and two (2) parts clean sharp sand. After rubbing, the surface must not be washed off but the ground up particles allowed to remain. When dry the surface will prove to be very light in color, hard, and the finish does not scale off.

(*b*) *Skilled Labor.*—The surface of concrete after it has become hard can be cut as natural stone would be cut, but this should be done by men who are experienced in stone cutting, and when treated in this way textures similar in character to those of natural stone can be obtained, in fact, the crandall, bush hammer, tooth tool, chisel and set can be used with such effect that at a short distance the appearance is identical with natural stone.

By cutting the material when green, it is impossible to get sharp corners. The round corners which are common in concrete are generally inartistic and crude in appearance. By allowing the material to become quite hard and using proper care, as sharp corners can be obtained as with natural soft stone such as limestone. It will thus be seen that the results to be accomplished have a controlling influence over methods to be used. In this way addi-

tions of concrete have been made to old buildings of natural stone with such a perfect matching of texture and color that it is impossible to tell the difference at 8 ft. distance.

It has been the experience of some engineers that just as cheap results can be obtained in bush hammering with common labor as by skilled. This, however, should be qualified again by the result to be obtained. On large surfaces where a coarse general treatment is desired and where the observer cannot get close to the surface, it can be tooled by common laborers when the material is quite soft. If, however, the observer can come within a few feet of the surface the above described treatment is not as good as cutting by skilled labor after the material has thoroughly set. A treatment used to a limited extent has been the splitting of blocks cast in a plane, so as to obtain a natural fracture, and a similar effect can be obtained by breaking off pieces of the surface by a stone set to obtain a natural fracture. In this manner the natural fracture of stone is more closely imitated, but as the whole general treatment is so unnatural to a cast material it is an inartistic treatment and is not recommended.

Still another way is by cutting in a lathe such objects as columns. After the cement has well set the stone can be cut in two without pulling out from the mortar. Thus the center of the stone can be exposed which is of large cross section relatively to the sand in the mortar, and a more stony appearance obtained. If the stone itself is capable of taking a polish a fairly smooth treatment can be given by rubbing it down in a lathe after turning. Mortar itself will not polish.

(*c*) *Sand Blast.*—A finish somewhat similar to that of scrubbing can be obtained by means of sand blast. It is important to use the correct size of nozzle, however, for this kind of work, and an exceedingly hard sand must be used, preferably that which has angular grains. A ½-in. diameter nozzle would produce very unsatisfactory results, because if an excess of cement be at the surface in one spot or on an edge as a joint between boards, its hardness would resist the cutting of the sand while softer portions on either side would be removed, thus emphasizing the ridge. Similarly a soft spot formed by a wide crack where the cement ran out leaving the sand behind, would be cut deeper by the large

nozzle. Experience has shown that a nozzle not over ⅛-in. diameter should be used so that it can be localized on small spots and that the nozzle must be held within a few inches of the surface to be cut if there is any variation in the density or hardness of the surface.

This method leaves the stone cleaner than with scrubbing and acid treatments.

(*d*) *Chemical Treatment.*—When the surface is chemically treated the materials used are usually commercial hydrochloric or nitric acid. The amount of dilution must be determined by experi-

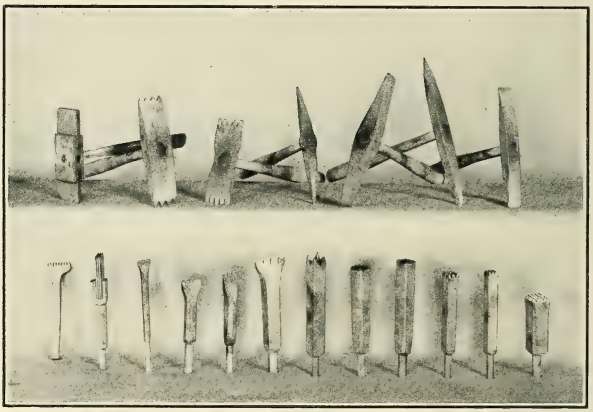

FIG. 1.—TOOLS USED IN DRESSING CONCRETE SURFACES.

ment and will vary with the age of the concrete. The older the cement the stronger must be the solution. For cement which is about 2 weeks old ordinary commercial acid should be diluted with 3 parts of water. When but a few days old a dilution of 1 part acid to 5 or 6 parts of water will be sufficient. A mixture of the two acids will produce stronger action than the use of one alone. They also produce a vapor which will soften the surface when exposed to it in a confined chamber and permit removal of the surface with very little scrubbing. After the scrubbing is completed great care must be used to remove all traces of acid, otherwise there will be permanent discoloration of the surface.

(*e*) *Age When Cut or Treated.*—The age of the mortar has a large influence upon the cost of the cutting and upon the texture of the surface. Good and economical results can be obtained when the cement is between 10 and 14 days old. It requires a fairly large job to justify the cost of setting up and operating a sand blast plant. Various building cleaning companies have portable outfits which are admirably adapted for this kind of work and which can be obtained in all large cities.

(*f*) *Tools. Kind, Quality, Temper.*—A new type of brush for rubbing or scrubbing green concrete surfaces which has proved very efficient in actual work is composed of ordinary wire mesh, such as fly screen, and is made by clamping together a sufficient number of sheets of wire, all of equal size, so that the exposed or rubbing edge will be at least 4 ins. wide. Such a brush has proved much more effective and cheaper than the ordinary wire brush.

Fig. 1 illustrates many types of tools used in dressing monolithic wall surfaces. Those in the upper row, also the one at left of lower row, are used for hand work; all the others fit into a compressed air hammer. The tools must be made of the best quality of tool steel as the concrete dulls them rapidly. The bush hammer shown at left is made with from 3 to 9 cuts to the inch; 6 cuts is as fine as is recommended. The weight is the same as in granite work. The multiple pick is 7 ins. long by 2 to 2½ ins. wide and weighs 2½ lbs. when new. The tooth chisel shown at left of lower row and the stone set which is similar in shape with a chisel edge, are used with a wooden mallet. They are the principal tools used in working soft natural stone and are handled in a similar way on concrete and cast stone. The other tools produce the same results with pneumatic power.

(*g*) *Effect of Cutting on Permeability.*—The waterproofness of concrete depends upon the density of the mortar. In the packing of mortar on surfaces which are to be exposed, greater care is usually taken in producing a dense uniform surface which would show a good appearance than in the packing of the whole mass of materials in the back. Therefore, before cutting, there is usually a dense skin of rich mortar on the surface which is practically impervious to moisture. When this is removed by any process the interior is more likely to absorb moisture than before. In one

particular job where the aggregate was composed of coral sand and coral stone which is very porous like cinders or blast furnace slag, after picking the concrete permitted water from a driving rain to pass through a 12-in. wall, whereas the wall previous to tooling was practically impervious. With a dense stone and quartz sand, a rich mixture, plastic consistency, well packed, a wall may be as impermeable after treatment as before. These facts are sometimes overlooked and cause unexpected and unpleasant results.

(*h*) *Effect of Cutting on Durability.*—A wall which will absorb and hold moisture is liable to disintegrate ultimately by the action of frost. If it is impervious to moisture the action of frost is merely an attack upon the surface and is very much slower, being practically the same as on any surface of a solid natural stone.

(*i*) *Effect of Cutting on Collection of Dirt.*—It is common sense to see that roughening a surface will permit a much faster collection of the dirt and dust blowing in the streets than a smooth and dense surface. However, as dirt and dust are usually so like the color of the cement itself, it is not objectionable, and these very fine particles can lodge in the pores of the concrete and assist in rendering them more impermeable. It is only where smoke is thrown out in large volumes that the smoothness of the surface must be considered to resist a change of color.

(*j*) *Advantages and Disadvantages of Certain Treatments.*—The advantages of the treatments classified under *a, b, c,* and *d* are that the surfaces being integral with the body of the concrete, are permanent, whereas when surface material is added as a coating it is not in every case permanent.

The surfaces treated by picking or tooling when the cement is hard, gain their appearance largely from the texture of the tool marks, and as the cement is not fully removed from the surface of the aggregate by the tools there is less life to the surface. When scrubbing and rubbing are done while the cement is green, this cleans the aggregate so that greater variety of color is obtained, and a smaller percentage of the exposed area is cement. This is also true of sand blasting, which cleans the aggregate of cement, but gives a flatter texture than cutting by means of tools.

With a chemical treatment there is the difficulty of completely

neutralizing the acid, which, if not completely accomplished, leaves its effect in color upon the surface.

(*k*) *Photographs.*

III. Coatings.

(*a*) *Texture and Condition of Masonry Surface to be Covered.*—Masonry as above used includes brick, terra cotta, natural stone, concrete, mortar and plaster. The covering coat must bond to the solid material of the masonry and not to the dust or dirt particles on the surface and the masonry should be sufficiently substantial to hold the covering. If the masonry is not substantial and rigid and contains cracks, the surface covering is pretty sure to crack over the cracks underneath.

(*b*) *Metal Lath. Kinds, Method of Erection.*—For outdoor purposes, it is essential to use galvanized metal lath. There are various kinds, woven, welded, expanded, any one of which can be used. That having a large cross section of metal and heavily coated with galvanizing material is likely to be the most durable, if moisture should penetrate the plastering to this material. It should be thoroughly tied with galvanized wire to furring at intervals not exceding 16 ins. The furring should leave sufficient space for the mortar to push through the mesh and clinch without interference from the backing to which the furring is attached.

(*c*) *Preparation of Base to Obtain a Bond.*—The texture should be as rough as possible with recesses in the surface which will support and key on a coating which is applied, and this surface should be free from dust and dirt which would prevent the bond or union. Before coating the surface the base should be thoroughly saturated with water so as to avoid suction of moisture from the covering coat, which would impair its strength and durability.

(*d*) *Material. Cement, Sand, Lime, Pigments.*—Materials used are cement, sand of a quality previously mentioned, also lime and pigments of various kinds and quantity.

In order to determine the relative quantities of hydrated lime and quicklime to be used in Portland cement mortars and concrete, a series of experiments were conducted upon samples of quicklime obtained from local dealers. These experiments consisted of adding

sufficient water to weighed quantities of quicklime to produce a paste. In another series of experiments sufficient water was added to slake the lime and reduce it to the consistency of cream, or to such a consistency that when a stick is immersed in the lime and withdrawn, the cream will drip from the end of the stick. This seems to be a very definite consistency, is easily determined and appears to be the best for the purpose.

It was found that the amount of water required to reduce the lime to a paste varied from 54 to 84 gals. of water per bbl. of quicklime of 300 lbs. The following represents the variations on the 8 samples of lime tested, the individual figures being 54, 59, 69, 71, 73, 73, 75 and 84 gals. of water per bbl. of lime. It is apparent that 5 of the 8 samples required between 69 and 75 gals., so that 72 gals. per bbl. would be a very fair average for these 5 samples.

To reduce these same samples of lime to a cream required a total of from 100 to 234 gals. of water per bbl. of lime, the individual figures being 100, 138, 149, 159, 161, 230 and 234. It is interesting to note that the lime which required the least water to make a paste, namely 54 gals., required the most water to reduce the lime to a cream, namely 234 gals.

These large variations in quantities of water required to reduce quicklime to a paste or a cream render it difficult to draw a specification applicable to any lime. It would seem, therefore, that such a specification can only state that where quicklime is used in place of hydrated lime three-quarters (¾) of the quantity of quicklime may be substituted, as 100 lbs. of quicklime when slaked will produce about 135 lbs. of dry hydrated lime.

(*e*) *Mixing.*—The mixing must be thoroughly done and the amount of water used is less than with mortars or concrete packed in molds, as otherwise the mortar would run from the lath.

(*f*) *Placing.*—The first or scratch coat should be composed of one (1) part Portland cement, two and one-half (2½) parts sand and one-half (½) part hair putty. This hair putty shall consist of long cattle hair or fiber, thoroughly worked into good lime putty. The second coat should be composed of one (1) part Portland cement, two and one-half (2½) parts of sand and with not over 15 per cent. lime putty. The third or finishing coat to consist of

one (1) part Portland cement to two (2) parts of sand, with not more than 5 per cent. of lime putty. (To this finishing coat may be added pebbles or other non-combustible material as may be desired to obtain a variation in the finished appearance.)

Where a total thickness of not more than one inch is required, it is practicable to apply it in two coats, *i. e.,* omitting the second coat above specified. In all cases, one coat should follow the previous one as soon as it has sufficiently set to allow of so doing. This will guarantee a bond of the two applications. The finished appearance may be varied by the materials used, both as regards color and size, by the mechanical method of applying or finishing, or by a combination of any of these methods.

(*g*) *Joints.*—Plastered surfaces are more subject to shrinkage cracks than monolithic work because a richer mixture is used and the thinness of the body of the material has less power to resist stress. Therefore, joints should be allowed at intervals where they will be as inconspicuous as possible, because as they are likely to occur anyway and are apt to be irregular in shape and position, they are unsightly to the eye. If the surface is troweled smooth they are likely to show in an objectionable manner. Unless the surface of the various coats is kept wet there is usually trouble from cracking. Care must also be taken to have a sufficient lap in the lathing; to have the lath well tied together and also to the furring so that there can be no settlement or sagging of the lath or the furring to which the lath is attached. The cracks can be filled with cement, by a pump spraying grout or run over with a tool which will fill them. They are somewhat sure to show on a smooth surface anyway.

(*h*) *Washes.*—Washes of cement or lime can be applied to plastered surfaces which will fill the fine cracks and give a uniform surface to the material. Washes are frequently applied to monolithic surfaces and material cast in molds and form a rather pleasing treatment when carefully and well done. Cement applied with a brush is not likely to give a pleasing result but is pretty sure to be streaked and irregular in color which is apt to be a muddy yellowish green rather than the gray of cement. If the wash is very thick, containing sand, and is plastered onto the surface and left in this condition or else lightly troweled with a wooden float, the

appearance will be much more uniform and of a natural cement color. This is the most usual treatment given to plain monolithic surfaces, like bridge abutments, foundations and mills in inconspicuous locations.

(*i*) *Paints.*—Paint, either linseed oil paint after special treatment of the surface or special cement paints containing no oil, can be used to good advantage in painting cement surfaces either monolithic or cast, and will give a pleasing appearance. The color is as much in the control of the designer as paints used on wood.

(*j*) *Enamel.*—Enamel paints may be used on cement if there is no opportunity for water or frost to get behind the enamel and cause it to peel off.

If the concrete surface is first treated it can then be enameled. A piece of cured concrete, thoroughly dry, is immersed for a few hours in a solution of one (1) part 40° Baumé sodium silicate and three (3) parts water, removed from solution, allowed to dry in warm place for 24 hours and again immersed in the same solution of sodium silicate, removed and allowed to dry in a warm place for 24 hours. It will be found that the concrete so treated will stand without dehydration of the cement a very much greater heat than untreated concrete.

Make a thin paste of the sodium silicate solution and red lead and paint the smooth surface of the concrete that has been treated with the solution of sodium silicate. Allow the paste on the concrete to dry for several hours, then place in a kiln or furnace and heat to 1600° F., and a fine glaze will develop. This glaze may be colored by adding mineral coloring matter to the lead paste.

(*k*) *Durability.*—There are in existence plastered surfaces which have lasted for 15 years and are still in a perfect condition. There are numerous cases of moisture getting behind a plastered surface applied to some solid back, as brick, terra cotta, stone or concrete, and there freezing, causing the plastered coat to peel. For this reason, plastered coats are not generally considered as durable or as desirable as a surface treatment which is monolithic with the body of the surface treated. If the surface is impermeable to moisture and there is no chance for moisture to get behind the surface from any source, the plastering should be durable.

IV. Ornamental Work. Possibilities and Limitations.

(*a*) *Cast Concrete.*—Concrete can be cast in molds of various kinds with a considerable degree of ornamentation. In order to fill the molds, an excess of mortar is necessary and the finish is largely that of the mortar, the stone being used in the concrete merely as a filling to reduce the cost. The stone can be exposed to view only in ornamental work where the surfaces are fairly plain and of large area.

(*b*) *Cast Mortar.*—Ornamental casting is largely confined to mortar. Much sharper detail can be obtained than with concrete, and even as sharp corners as are usual with limestone. Where sharp corners are not required and the surface is cut, scrubbed or sand blasted as previously described, colored sands or stone grit can be used to produce pleasing color effects.

(*c*) *Plastered Mortar.*—Plastered surfaces can be placed on rough backs with good results. This is possible only with a mortar of a rather fine aggregate and therefore cannot give as large a variety of surface textures or colors as the cast, and the ornamental effects are largely those of straight molded members, as in a cornice, which can be run by the plasterer in place. These, however, are cheaper in cost than those cast and for this reason are quite extensively used.

(*d*) *Kinds of Molds.*—*Wood.*—For rigid molds, wood is chiefly used and ornaments can be cast in any form in which wood molds are practical, except that no undercut work can be done as to remove the wood would break the mortar. When wood is used it must be carefully filled with paint, oil, shellac, soft soap or other suitable material, so as to prevent the absorption of moisture from the mortar which would carry with it some of the cement bonding the mortar to the wood and thus destroy the face of the casting upon removal of the mold.

Iron.—Iron molds are used to some extent but are too expensive unless a great many castings are to be made exactly alike. It is not economical to make elaborate ornaments with iron molds on account of the expense of the mold. Sometimes stamped sheet metal can be used as a mold for the face of the casting but the appearance is not pleasing and is seldom desirable.

Plaster.—Plaster of Paris is very extensively used and by cutting it into small pieces and putting them together, can be used for slightly under-cut work. Plaster of Paris molds can be used for quite elaborate castings and for castings containing as much as 2 or 3 cu. yds. of concrete. As the plaster is not very strong in itself it is reinforced by burlap or some other fibrous material. Its surface must be carefully treated to make it non-absorbent to moisture, as otherwise it will be damaged by the material cast in it. Each time a casting is made the surface must be again prepared to resist the attack of moisture. These molds are rather expensive on account of the high class of labor necessary to make them. It is usually necessary to make a model before the mold can be made.

Glue.—Where a considerable amount of undercut work is required molds are made of glue. This requires considerable care and skill. The best quality of gelatine glue is required, mixed with the minimum amount of water necessary to dissolve and melt it at a low temperature. Around the model is placed a plaster form to receive the glue, which is poured in about 1½ in. thick between the model and the form. The glue is then cut in sections so that it can be removed from the finished casting, is removed from the model and re-assembled. It must be used before it has had time to dry up. Its surface should be coated with lard oil to prevent the absorption of water, which would disintegrate its surface. The lime of the cement also attacks the glue and decomposes it. For these two reasons castings of glue must be made with a minimum amount of moisture. Not more than three castings can be made before the glue will have to be re-melted and re-cast, and after three or at the most four molds it will be so impaired in quality that it must be thrown away and new glue used. However, such undercut work can be done with glue molds and it is frequently necessary to use them.

Sand.—Sand, either plain or treated with some material such as clay, loam, paraffine or acid, is used for molding purposes. The sand must be fine, so as to hold its shape, like sand used in iron foundries, and can be used for any object from which the pattern can be drawn. In fact, as elaborate objects can be cast in sand as in metal. As the sand can be used over and over again, the expense of its use is determined by the elaborateness of the object which

determines the amount of labor necessary to make the mold. A very promising departure in concrete molding is the use of sand and paraffine as a molding material. At 150° or above a little paraffine will disseminate itself through a great deal of sand. This sand shows the presence of paraffine very little and can be molded at 110° or thereabout, *i. e.*, while slightly warm to the touch. Below 90° this material becomes quite rigid, and would seem for concrete casting to be ideal. Molds made of Portland cement and mortar have been used with success on plain work. It is the experience of one company having had years of practice in all kinds of cast work that absorbent molds into which is cast semi-liquid concrete give the most uniform color and texture and produce concrete of 10 to 25 per cent. greater density than when tamped, pressed or poured into non-absorbent molds.

V. Waterproofing.

(*a*) *Necessity.*—With thoroughly mixed and carefully placed concrete made from materials properly graded for size so as to obtain maximum density with mixtures not exceeding 1-2-4, the resulting concrete would be so impermeable that waterproofing is unnecessary. With leaner mixtures or less care in the grading of materials the concrete will be somewhat porous, rendering necessary in many cases the waterproofing of the surface or in order to prevent penetration of moisture, the waterproofing of the mass when it is cast.

(*b*) *Mixed in Concrete.*—It has been found by experiment that the percentage of water added to concrete in mixing has a very marked effect upon its permeability, and that even slight variations from the proper consistency cause very much greater variations in permeability. It is, therefore, of great importance that concrete be mixed to the proper consistency to obtain the best results. As the best percentage of water varies with different cements and aggregates, no rule can yet be laid down, but with any given cement and aggregates preliminary tests may readily be made to determine the proper amount of water to use in mixing. This consistency can be gauged approximately by the eye. It is such a consistency that water flushes to the surface freely when the concrete is tamped,

but not before tamping, and when thoroughly tamped the mass just barely quakes. For further information see paper on "Waterproofing with Water."*

The integral method of waterproofing by means of a powder paste or liquid added to the mass of the concrete, has many advocates and is probably the most logical method of waterproofing where it can be done. There are many proprietary preparations which are claimed to accomplish the desired results and many trials and tests have been made to determine their efficiency. The list of those so far tested is as follows:

INTEGRAL WATERPROOFING COMPOUNDS.

Manufacturer.	*Trade Name.*
American Diamond Blast Company, 50 Church Street, New York City.	Anti Hydro.
Aquabar Company, 1228 Locust Street, Philadelphia, Pa.	Aquabar.
Ceresit Waterproofing Company, 103 Park Avenue, New York, N. Y.	Ceresit (paste).
Chesebrough Manufacturing Company, 17 State Street, New York, N. Y.	Petrol Oil.
Glidden Varnish Company, Cleveland, Ohio.	Special Material Nos. 1 and 2.
Goss and Company, 334 Fifth Avenue, New York, N. Y.	Medusa Compound.
Harrison Waterproofing Materials Co., 30 Church Street, New York, N. Y.	No. 100 Cement Waterproofing Powder
Hercules Waterproofing Cement Co., Buffalo, N. Y.	Hercules Waterproofing Compound.
Horn and Company, A. C., 8 Burling Slip, New York, N. Y.	Hydrotite.
Ideal Concrete Machinery Company, South Bend, Ind.	Ideal Waterproofing Filler.

* *Proceedings*, Vol. VII, p. 672.—ED.

Manufacturer.	*Trade Name.*
Impervious Products Company, 38 West 28 Street, New York, N. Y.	Impervite.
Keystone Paint and Filler Company, Muncey, Pa.	Keystone Waterproofing Compound.
McCormick Waterproofing Paint Co., St. Louis, Mo.	McCormick Waterproofing Compound.
Marvel Waterproofing Company, 14 Church Street, New York, N. Y.	Marvel Compound No. 1 (paste).
Maumee Chemical Company, 403 St. Clair Building, Toledo, O.	Waterproofing Compound.
Trussed Concrete Steel Company, 1 Madison Avenue, New York, N. Y.	Trus-Con. Waterproofing Paste.
Toch Brothers, 320 Fifth Avenue, New York, N. Y.	Toxement.

The general conclusion seems to be that while some of these materials have considerable virtue, they cannot be depended upon. Too much depends upon the manner in which they are used and the thoroughness with which they are mixed with the concrete.

One of the most common materials used for waterproofing with considerable success is lime. It can be thoroughly slaked and mixed with concrete as a putty, or better, thinned to a whitewash. Another form producing similar results is hydrated lime, which is a common lime slaked, dried and pulverized to a fine powder, and mixed dry with the mortar.

(*c*) *Method of Incorporation.*—With lime an amount equal to 5 per cent. of the cement by weight will give good results without impairment of strength in the resulting concrete. If mixed dry it should be carefully sprinkled over the sand and cement and so thoroughly mixed by hand or machine as to obtain a uniform distribution throughout the mass. If mixed wet the putty should be so thinned that it can be as easily and uniformly distributed through the whole mass as the water.

The Committee has made some further study of the waterproofing compounds on the market which are to be incorporated in the concrete at the time of mixing, and is of the opinion that in the results obtained with this class of materials, workmanship counts

for more than the compounds themselves. Whether the waterproofing ingredients be colloidal clay, lime or a patented powder, paste or liquid, it must be most thoroughly and uniformly mixed throughout the mass of the concrete, and this can only be done well in a batch mixer and by leaving the charge in it an extra length of time. With carefully selected and graded materials, thoroughly mixed and skilfully placed, concrete is in itself waterproof without the addition of foreign matter.

(*d*) *Surface Treatment Without Changing Color.*—An investigation has been made of many of the compounds now on the market which do not greatly alter the color of the surfaces to which they are applied. While many of these preparations give good results soon after they are applied they do not seem to be able to endure exposure to the weather. Many of them will show very satisfactory waterproof qualities when first applied, but lose their value in a few months of exposure to the elements.

The following list alphabetically arranged, comprises all the compounds the Committee has investigated to date; 2 compounds stood up very well when first applied but gradually lost part of their efficiency upon exposure to the weather; 10 compounds did well when fresh but rapidly deteriorated. The remaining 27 did not prove so effective when first applied, and on exposure lost most of the virtue they did have.

COLORLESS WATERPROOFING COMPOUNDS.

Manufacturer.	*Trade Name.*
Billings, King and Company, 438 Pearl Street, New York, N. Y.	Concrete Waterproofing Paint.
Damarra Manufacturing Company, 32 Broadway, New York, N. Y.	Cement Covering and Waterproofing Transparent. Cement Waterproofing. P. 1. Solution.
Electric Cable Company, Bridgeport, Conn.	Liquid Voltax No. 175. Liquid Voltax, No. 103.
Flexible Compound Company, 3607 Haverford Ave., Philadelphia, Pa.	Flexible Compound.
Glidden Varnish Company, Cleveland, Ohio.	W. P. Anti-Stain Preservative Liquid Waterproofing. Transparent Waterproofing.

Manufacturer.	*Trade Name.*
A. C. Horn Company, 8 Burling Slip, New York, N. Y.	Dehydratine No. 2.
Hydrolithic Cement Company, 138 Jackson Boulevard, Chicago, Ill.	Liquid Diamond Waterproofer.
Illinois Dampproofing Company, 145 LaSalle Street, Chicago, Ill.	I. D. P. Transparent.
Kessler Fluats Company, 43 West 27 Street, New York, N. Y.	Magnesium Fluats Encaustique.
Keystone Paint and Filler Company, Muncey, Pa.	Keystone No. 3. Transparent Waterproofing.
Marvel Waterproofing Company, 14 Church Street, New York, N. Y.	Marvel Compound No. 10. Marvel Compound No. 40.
Miller and Company, Clifford, 110 East 23 Street, New York, N. Y.	Minwax Clear Waterproofing.
National Waterproofing Company, 607 Harvester Building, Chicago, Ill.	Te-pe-co Waterproofing.
North Jersey Paint Company, St. James Building, New York, N. Y.	Edison Waterproofing Paint. Anhydrol.
Preservative Products Company, 1133 Broadway, New York, N. Y.	Esco Preservative. Neutral No. 1. Neutral No. 2. Neutral No. 3. Impervo.
Sherwin Williams Company, 66 Broadway, New York, N. Y.	Weatherproof Coating.
Standard Preservative Company, 136 Liberty Street, New York, N. Y.	P. P. P. Cement Floor Finish.
Standard Waterproofing Company, 720 W. Washington St., Indianapolis, Ind.	Standard Waterproofing.
Sterling Varnish Company, Pittsburgh, Pa.	No. 7 Special Waterproofing Composition.
Toch Brothers, 320 Fifth Avenue, New York, N. Y.	Toxloxpore. Cement Filler.
Trussed Concrete Steel Company, Trussed Concrete Bldg., Detroit, Mich.	Trus-Con Liquid Waterproofing.
Unknown.	Egyptian Preservative.
Virginia Waterproofing Corporation, Norfolk, Va.	Coleman's Waterproof Filler.

Manufacturer.	*Trade Name.*
Wadsworth, Howland and Company, 83 Washington Street, Boston, Mass.	Bay State B. & C. Waterproofing.
Zibell Damp Resisting Paint Company, 273 Water Street, New York, N. Y.	Zibell Compound.

The Committee invites the manufacturers of all waterproofing compounds to submit samples of their product for investigation. It is the hope of the Committee that further search will show that there are products of this nature which will not only render concrete walls dampproof, but will keep them so for a considerable length of time.

One more treatment can be mentioned at this point, namely the use of paraffine, which is a staple material of mineral origin. This can be applied to the surface hot with a brush, or what is still better, the surface can first be heated so as to permit the hot paraffine which is later applied to soak to greater distance into the surface pores before it has time to cool and harden. This material will give a glossy, oily appearance to the surface, if used to excess, which should be avoided for this reason.

(*e*) *Surface Treatment Changing Color.*—These treatments may be divided into two classes: *First,* those which give white or light tints or other colors pleasing to the eye, and are of a decorative nature. *Second,* those which employ compounds of tar and asphalt, which are necessarily black or nearly so, and are, therefore, seldom used on exposed surfaces.

During the past two years there have appeared on the market a large number of concrete paints which are claimed to be waterproof. The Committee has investigated a number of these, and finds them much more efficient than the colorless solutions above referred to.

The presence of a finely divided pigment serves to seal up the small pores in the surface of the concrete, leaving less work to be done by the vehicle. In most cases the proportion of pigment used is small, and by using a cement color but little change in the surface appearance of the concrete needs be made unless desired, while, on the other hand, almost any shade or color wanted may be obtained. In this way the waterproofing coating is made more serviceable, and at the same time ornamental. All of these treat-

ments are applied in the same manner as paint, and are probably as durable on concrete as paint is on wood.

Paints.—Linseed oil paints should not be applied directly to new concrete, or to any concrete which has not been long exposed to the weather. The free lime in concrete not thoroughly weathered out by exposure to the elements will saponify the oil, destroy its adhesive power and its life, and the paint will soon scale off. After concrete has been exposed for a long time to the elements, linseed oil paint may be used with greater safety. It is difficult, however, to determine when the concrete is sufficiently free from lime to render it safe for the application of linseed oil paints, and it is therefore safer to avoid their use directly on concrete.

There are preliminary treatments which may be given to concrete which render it harmless to linseed oil paints. The best known, and perhaps the most thoroughly proven treatment of this kind is that devised by Mr. Chas. MacNichol,* Washington, D. C. It consists in giving the concrete, after it has thoroughly set, a priming coat of a solution of zinc sulphate, 8 lbs. to a gallon of water. This is applied with a brush, and after drying, ordinary linseed oil paints may safely be applied over it. This process has been in use for many years with uniformly satisfactory results.

There are on the market many paints designed especially for use on concrete, which may be applied directly to new work with more or less durable results, yet an alkali proof size is a wise precaution. Some of these concrete paints will not stand wet weather for any great length of time, and others have a tendency to scale off.

The following paints are being tested by the Committee:

Outside Waterproofing Coatings (Colors, Except Black).

Manufacturer.	*Trade Name.*
Billings, King and Company, 438 Pearl Street, New York, N. Y.	Driwall (various colors).
Cabot, Samuel, Boston, Mass.	Waterproof Cement Stains (Various receipts.)
Dexter Brothers Company, Boston, Mass.	Petrifax.

* See *Proceedings*, American Society for Testing Materials, Vol. X, p. 396.—Ed.

Manufacturer.	*Trade Name.*
Fahsel, C. D. 250 Audubon Ave., New York, N. Y.	Pineoleum.
Goheen Manufacturing Company, Canton, Ohio.	Concrewaltum.
Hoyt, C. H., Cleveland, Ohio.	Durable Lac Waterproofing Coating.
Ideal Concrete Machinery Company, South Bend, Ind.	Billings-Chapin and Company's Duck Brand.
Illinois Dampproofing Company, 145 LaSalle Street, Chicago, Ill.	Protex.
Insulite Chemical Company, Aurora, Ill.	Insulite red and green.
Ironite Company, 84 LaSalle Street, Chicago, Ill.	Ironite.
Koat-O-Stone Company, 227 Greenwich St., New York, N. Y.	Flatcoat. Koat-O-Stone.
Marvel Waterproofing Company, 14 Church Street, New York, N. Y.	Marvel Compound No. 80.
Moore and Company, Benj., 231 Front Street, Brooklyn, N. Y.	Lime Proof Paint. Mooramel.
Miller, C. L., 110 E. 23 Street, New York, N. Y.	Minwax Waterproof Coating
Murallo Company, 50 Church Street, New York, N. Y.	Irono Waterproof Paint.
Patton Paint Company, Newark, N. J.	First Coat Cement Paint. Cementhide (inside and outside).
Rinald Brothers, Philadelphia, Pa.	Flat Unicoat.
Toch Brothers, 320 Fifth Avenue, New York, N. Y.	Liquid Konkerit.
Trussed Concrete Steel Company, 1 Madison Ave., New York, N. Y.	Outside Wall Finish (various colors).
United States Gutta Percha Paint Co., Providence, R. I.	Rice's Flat White. Rice's Granotith.
Universal Compound Company, 449 W. 28 Street, New York, N. Y.	Gray Morganite.

Outside Waterproofing Coatings (Black).

Manufacturer.	*Trade Name.*
Insulite Chemical Company, Aurora, Ill.	Insulite Black.
National Insulator Company, 39 Sudbury Street, Boston, Mass.	Nico Insulation and Waterproofing Paint.
Semet-Solvay Company, 100 William Street, New York, N. Y.	Solvay Concrete Coating (grade S). (grade H).
Standard Paint Company, 100 William Street, New York, N. Y.	S. P. C. Dampproofing Paint.
Toch Brothers, 320 Fifth Avenue, New York, N. Y.	R. I. W. No. 232. R. I. W. Marine Cement.
Tuttle Roofing Company, 522 E. 20 Street, New York, N. Y.	Antihydrine.
Universal Compound Company, 449 W. 28 Street, New York, N. Y.	Exhydroplastic.
Warner, Wm. D., 50 Exchange Place, Providence, R. I.	Metalcotine.
Wilkinson Company, E. De F., 48 Exchange Place, Providence, R. I.	Standard Coating.

The above list is probably quite incomplete, and if the makers of such paints will co-operate with the Committee, the list will be extended to cover all commercial exterior concrete paints.

(*f*) *Mortar.*—For many places where a surface must be waterproofed a rich mortar carefully applied by skilled workmen will not only answer the purpose of a finished appearance but will resist a water pressure of a number of feet head. It is a treatment with which good results can only be obtained by experienced and skilful workmen under intelligent supervision.

VI. Limitations, Defects, Blemishes of Various Kinds, and Remedies.

(*a*) *Range of Colors.*—The range of color in mortar by use of pigments is limited to red, yellow, blue and black. No other satisfactory color has come within the observation of the Committee. Red and yellow can only be used as faint tints, as in

quantities exceeding 5 per cent. by weight they seriously impair the strength. Blue shades are obtained by using a small quantity of lampblack, whereas a large quantity produces a black color. The range of color in the aggregate is limited to the colors found in natural stone or other substances as previously mentioned, and which are exposed later by the removal of all cement from the surface of these particles of aggregate.

(*b*) *Variation of Color.*—With the admixture of pigments, unless extremely thoroughly mixed, there will be a variation in the color of the mortar. The great difficulty, however, is the variation in color between the different batches, due to a slight error in proportioning, the variation in the quality of the pigment, or the treatment during setting and hardening. Cement setting in a hot dry place will have a lighter shade than when setting slowly in a wet or damp place, even if all other conditions of manufacture are identical. Where the color is obtained from the aggregate, variation is due to a variable distribution throughout the mass or in placing an excess of stone in one part of the mass over that in another.

(*c*) *Craze Cracks, Checking.*—This is one of the most neglected subjects with which the manufacturers and users of cement have had to deal. There is little question that checking is caused by differential expansion and contraction, but years of continued and vigilant investigation have not furnished data from which definite conclusion as to the real cause of crazing in concrete can be obtained. The theory that cement when cured in air contracts, and when cured in water expands, advanced by many, does not satisfy all the phenomena which have been observed in experiments. It is quite easy to prove that any concrete, irrespective of cement and aggregate, is never entirely free from crazing. It is well known that the greater the amount of cement the more the crazing, also that careful curing greatly reduces it. With a view to bringing out discussion of this all important subject the Committee takes the liberty of appending some of the phenomena observed in experiments, and conclusions reached, as follows:

1. That all Portland cement mortar has a tendency to craze. The smoother the concrete the more marked are the craze lines, for the reason that the dirt accumulating in them makes them

apparent, and that a porous concrete absorbs more of this dirt than a dense concrete, but uniformly over its surface. It is only the fact that the dirt collects in the craze cracks that makes them apparent on the denser material.

2. That there is quite often an exudation from cement concrete which collects in craze cracks, which when analyzed is found to contain practically all the soluble salts which exist in Portland cement mortar, showing that more or less of this material is absorbed and carried to the surface by the moisture when the concrete dries.

3. Exposure of green concrete to draughts, sun and rain, appreciably increases the amount of checking, but no matter how carefully concrete is protected or for how long a period, it will eventually craze and check on exposure. This has been proved by curing concrete in water kept at very nearly a constant temperature for a period of two years, and that casts so made when sawed in half, thoroughly dried out, and exposed to the air, checked inside of thirty days. Aggregates have an influence on crazing. Marble and feldspar check much more rapidly than trap or granite.

4. That such crazing or checking is not confined entirely to the surface of concrete.

5. That the addition of waterproofing compound does not stop checking, but in hindering the concrete from absorbing dust and dirt prevents the crazing from showing temporarily.

6. The amount of cement necessary to produce dense concrete may be reduced by properly proportioning the aggregates and by careful mixing, which will materially decrease the tendency to checking and crazing.

(*d*) *Expansion and Contraction Cracks.*—Such cracks are due almost entirely to thermal stresses and can be overcome and controlled by proper reinforcement in lengths of concrete up to 500 ft. and perhaps longer, but as such should be treated under the subject of concrete design.

(*e*) *Irregular Size of Castings.*—The prevention is stronger bracing of molds; the correction, chipping before or after the work is hard. This is a frequent point for criticism against appearance of concrete. It is not a blemish when the work is well done any more than the mortar joints in brick and stone work. It may be

minimized by cutting back the surface of the work before it is hard and grouting on a new one.

(*f*) *Mortar and Stone at Horizontal Joint between Days' Work in Concrete Wall.*—In building monolithic walls where work is stopped for the day, an excess of mortar is frequently tamped to the surface and sets. The following day when new concrete is added an excess of stone settles through the mortar to the hard surface below, so that after cutting the surface a line of mortar is shown about a ½ in. wide, with an excess of stone directly above. This is avoided by scraping off a ½ in. of the surface when the day's work is done before the concrete has had time to set and by care in using a stiff mix in the first batch deposited the next day, so that the stone is held uniformly distributed in the mortar which is too stiff to permit the stone to settle through it.

(*g*) *Patches.*—The darker color seems to be caused by using a richer mixture, steel rubbed off from trowel with which applied, less surface evaporation from patches, or bringing to surface by troweling neat cement particles or the aggregates. It has been conclusively demonstrated that a patch cannot be detected, even after several years of exposure, if applied with due care and by frequent addition of some material to lighten the color of the materials put into the patch.

(*h*) *Efflorescence. Cause and Remedy.*—Efflorescence is produced by the surface evaporation of water carrying dissolved salts from the interior of the mass. It is not observed where a wall is always dry. Portland cement and sometimes the aggregates contain soluble salts which are brought to the surface in solution by water and left there by its evaporation. The remedy is to prevent water from penetrating the mortar or concrete. If this tendency is very slight, as for instance in the wall of a building which is somewhat porous, allowing a driving rain to soak into it and to later percolate to the surface, bringing with it in solution soluble salts, the appearance of efflorescence can be very much reduced or practically removed by waterproofing the surface with some compound which will prevent all absorption and percolation whatsoever.

(*i*) *Porosity. Cause and Remedy.*—Porosity is a lack of density. The causes may be too little cement to fill the voids, too much fine sand, not enough fine sand (*i. e.*, sand not properly

graded), stone not of proper size or properly graded for the work, not sufficient water added to the mixture to lubricate it and feed the cement.

(*j*) *Frozen Surfaces.*—Freezing retards the setting of cement. Alternate freezing and thawing will destroy the bond of green concrete. If concrete of proper density has attained a proper strength it is unaffected by freezing action. Where green concrete is subjected to alternate freezing and thawing it may result in a scaling of the surface.

The remedy after the surface is frozen and where the material scaled off does not have to be replaced, is to re-surface that remaining in place. If the material must be replaced the texture of the surface originally desired must determine largely the method of treatment. If this be a plastered surface it is a simple matter to roughen up the back to give a key for plaster and to place it on. If it is to be a tooled concrete surface, then a sufficient mass must be removed to enable the placing of a new facing containing aggregate of the same size and proportions as the original work, so as to build out the face and to provide sufficient strength and stability either to stand by itself or to be properly keyed to the back by suitable methods such as the use of reinforcement.

(*k*) *Dusting.*—This trouble is due to soft sand, unsuitable material, too little cement in mixture, bad mixing, too much time elapsing between mixing and finishing, finishing mixture too dry or too wet, use of "dryers" (*i. e.,* dry cement with or without sand to dry out the surface so that it can be quickly finished), or permitting the surface to dry so rapidly as to impair the setting of the cement. When a thin surface is placed on one which has become hard there is liable to be dusting, because there is no absorption by the concrete to take care of the excess moisture in the mortar. It, therefore, must come to the surface and in so doing it brings with it the soluble salts of Portland cement, called laitance, and some of the finest particles of the cement, the impalpable powder, both of which are easily brushed off. Again, because the moisture rises the surface cannot be troweled as soon as it ought to be, and so long a time has elapsed that the setting of the cement is broken up or impaired, thus materially reducing its strength. The remedy, where color is not an object, is the application

of one or two coats of well boiled linseed oil. Paint is sometimes used, but is not a satisfactory material because as a rule it does not harden the dusty surface and the dust prevents its bonding to the mass below so that it scales off, leaving an unattractive appearance. In some cases the top may be ground off until a hard surface is obtained, as in terrazzo work.

VII. Costs.

The cost of unskilled labor for picking or rubbing concrete when 6 to 24 hours old is about 1 ct. per sq. ft. If 2 days old, so that fiber brushes and water have to be used the cost runs from 2 to 3 cts. for actual dressing of the surface. In order to be able to remove the forms from the surface at these ages, they will have to be designed in such a way that the price may vary from nothing to 5 cts. per sq. ft.

For a special facing material of 1 in. thick of one (1) cement, one and one-half (1½) sand and two and one-half (2½) of fine crushed stone or pebbles, placed at the same time with a coarse backing in the walls, will add about 3½ cts. per sq. ft. to the cost of the surface. This cost is independent of the method of treating the surface later.

Tooling with multiple picks by common labor when the surface is green will cost from 3 to 4 cts. per sq. ft.; when hard, from 5 to 10 cts.

The cost of the use of pneumatic tools varies materially with the size of the job and to what extent the work is scattered. For long low walls pneumatic tools will cost more than hand. For large areas like the soffit of a big arch, pneumatic tools will be cheaper than hand work by one-half to one-third.

Sand blasting when the cement is 1 to 2 weeks old will cost about the same as bush hammering; when older it will cost considerably more.

Extended experience gives the following cost data: For hand tooled cast concrete four or six grooves to the inch with steel tools, the cost on a contract of tooling some 10,000 sq. ft., with skilled labor, was 15 cts. per sq. ft.; the same kind of tooling with pneumatic tools about 12 cts.; the same tooling done in a factory by grinding with a special designed machine, 1 ct. This special

designed machine consists of cutting wheels suspended on an arm like a pendulum wood-working saw revolved at a high rate of speed, so that concrete of almost any age could be cut without disturbing the stone in the aggregate. The cost of crandalling, being an average of many jobs, with skilled labor, is about 10 cts.; with unskilled, about 5 cts.

VIII. Specifications.

The Committee submits the following specifications:

(*a*) *Scrubbed Concrete Surfaces.*—The proposed specification* (see p. 594) has been used in essentially the same form and has assured the best of results.

(*b*) *Cement Stucco.*—The proposed specification (see p. 586) is submitted for discussion. It is earnestly requested that the same be carefully studied and suggested changes should be sent to the Association as soon as possible, but not later than December, 1911.

L. C. Wason, *Chairman.*
Cloyd M. Chapman,
Albert Moyer,
H. H. Quimby,
E. J. Russell.

LIBRARY

* Adopted by letter ballot, September 18, 1911.—Ed.

National Association of Cement Users.

PHILADELPHIA, PA.

PROPOSED STANDARD SPECIFICATIONS FOR PORTLAND CEMENT STUCCO

Materials.

Cement.—The cement shall meet the requirements of the Standard Specifications for Portland Cement of the American Society for Testing Materials, and adopted by this Association (Standard No. 1).

Fine Aggregate shall consist of sand, crushed stone, or gravel screenings, graded from fine to coarse, passing when dry a screen having ⅛-in. diameter holes, shall be preferably of silicious materials, clean, coarse, free from loam, vegetable or other deleterious matter, and not more than 3 per cent. shall pass a sieve having 100 meshes per linear inch.

Lime.—The lime shall be thoroughly hydrated by the manufacturer. If hydrated by the contractor, it shall be slaked in two and one-half (2½) volumes of water and allowed to stand at least one (1) week before using.

If quick lime slaked on the job is used in place of dry hydrated lime, three-fourths (¾) the quantity by weight of dry lump lime shall be taken, or 35 lbs. of dry lump lime to each barrel of Portland cement.

Hair or Fibre.—There shall be used only first quality long cow hair, free from foreign matter, or a long cocoanut fibre well combed out.

Coloring Matter.—Only mineral colors shall be used, but no

coloring matter which is affected by lime, Portland cement or the elements is permissible.

Water shall be clean, free from oil, acid, strong alkalies or vegetable matter.

Mixing.

The ingredients of the mortar shall be thoroughly mixed dry to a uniform color, sufficient water added to obtain the desired consistency, and the mixing shall continue until the cement is uniformly distributed and the mass is uniform in color and homogeneous.

The lime, when slaked on the work, shall be diluted to a thin creamy consistency and shall be added with the water. When dry hydrated lime is used it shall be mixed with the dry mass before addition of water.

The hair or fibre shall be added during the process of wet mixing.

a. Measuring Proportions.—Methods of measurement of the proportions of the various ingredients, including the water, shall be used which will secure separate uniform measurements at all times. All proportions stated are by volume.

b. Quantity.—There shall not be mixed at one time more mortar than will be used within one (1) hour. Mortar which has begun to stiffen or take on its initial set shall not be used.

c. Hand Mixing.—The mixing shall be on a water-tight platform and the materials shall be turned until they are homogeneous in appearance and color.

d. Consistency.—The materials shall be mixed so as to provide sufficient water to insure a proper bonding and a dense mortar free from voids.

e. Retempering.—Retempering mortar, *i. e.*, remixing with water after it has partially set, shall not be permitted.

Preparation of Original Surface.

Lath.

Sheathing Boards.—They shall be not less than 6 ins. or more than 8 ins. wide, dressed on one or both sides to a uniform thickness of not less than $\frac{7}{8}$ in. They shall be laid diagonally across the wall studs and fastened with two nails at each stud.

Sheathing Paper.—Next to the sheathing boards shall be laid in horizontal layers beginning at the bottom, a substantial paper well impregnated and thoroughly waterproofed with tar or asphalt. The bottom strip shall lap over the base board at the bottom of the wall, and each strip shall lap the one below it at least 2 ins. The paper shall lap over the grounds and flashings at all openings.

When required, the lower horizontal edge of each strip shall be cemented with hot or liquid tar or asphalt compound, to the strip below and to the grounds and flashings at all openings. All tacking shall be within 2 ins. of the top horizontal edge, where tacks will be covered by the lap of the strip above.

Furring Strips.—Strips of galvanized metal at least ¼ in. high shall be placed vertically over the sheathing paper not more than 16 ins. apart, held in place by as few galvanized staples as are necessary. When the studding of the wall is spaced 16 ins. on centers, the furring strips shall be placed directly over them, and in any case they shall be placed as nearly as possible over the studs. The furring strips shall be of crimped sheet metal ribbon, not thinner than No. 18 gauge, galvanized after cutting to width, or of other shape of galvanized metal giving equal results.

Around curved surfaces the furring strips shall be spaced sufficiently close together to hold the lath not less than ¼ in. or more than ¾ in. from the sheathing board.

Furring strips shall be placed not less than 1½ ins. or more than 4 ins. on each side and below all window and door and other openings.

Furring strips shall be provided at all vertical joints in the metal lath.

Metal Lath.—Over the furring strips shall be placed a galvanized metal lath of woven wire or expanded or deformed sheet metal. If of woven wire, it shall have meshes not less than ¼ in. in their narrowest dimension and not greater than 1 in. in their largest dimension. The woven lath shall be thoroughly galvanized after weaving.

If the lath is of expanded or deformed sheet metal, it shall be of such a form that when plaster is applied to one side when in place against the sheathing boards, the plaster may easily be pressed through the lath against the boards so as to completely

surround and embed the lath, leaving no air space or uncovered portions. No further cutting, expanding or bending shall be done after galvanizing except the cutting and bending necessary to fit the lath on the job.

The metal lath shall be stapled to the sheathing boards with galvanized wire staples, or drive hooks, straddling the furring strips, and placed not more than 8 ins. apart. They shall be of such length as to extend to the inner surface of the sheathing boards.

Metal lath shall be placed with its stiffer axis horizontal, and each sheet shall lap at least one mesh, or rib, or row of deformations as the case may be. Lath of small mesh or openings shall be so lapped as to permit the plaster being troweled through and completely embedding it. If the stiffer axis is along the narrow dimension of the sheet so that the sheets are placed vertically on the wall, then they shall be of such width that all joints between sheets shall come over the furring strips.

Wood Lath.—Over the sheathing paper there shall be placed vertically, 8 ins. on centers, No. 1 spruce, cedar or white pine laths, each fastened with 4 lathing nails. There shall then be applied the same quality lath in horizontal layers, spaced ½ in. between laths, and fastened with one galvanized nail to each furring strip. When in place and before applying mortar the lath shall be thoroughly saturated with water from a hose, applied with sufficient force to wet the lath on both sides and sufficient time shall elapse before applying mortar to permit the lath to absorb the water.

Immediately after application of the first coat the surface shall be scratched and covered with light weight 1 in. mesh galvanized chicken wire fastened with galvanized staples of sufficient length to penetrate the sheathing boards. The wire shall not be cut at corners but carried at least 1 ft. beyond.

BRICK OR TILE.

Existing surfaces to be stuccoed shall have all loose, friable or soft mortar removed from the joints to a depth of not less than ½ in. All dirt, dust or any other foreign matter shall be removed by means of a wire brush, stiff broom or other effective means. In case the surface has been painted, is oily or otherwise in such condition that the stucco will not firmly adhere, then metal furring and lathing shall first be applied.

New surfaces shall have ample roughness to assure a strong bond and key between the stucco and the surface. The mortar joints shall not be less than $\frac{3}{8}$ in. thick and the mortar shall be omitted from or raked out of the joints for at least $\frac{1}{2}$ in. back from the face to which the stucco is applied. Before placing the scratch coat the surface shall be brushed clean from all dust, dirt or loose particles.

SCRATCH COAT.

Proportions.—The first or scratch coat shall contain one (1) part Portland cement, two and one-half ($2\frac{1}{2}$) parts fine aggregate and one-fourth ($\frac{1}{4}$) part dry hydrated lime.

For application to metal or wood lath sufficient hair or fibre shall be added to the mortar to cause a good bond with the lath. As little as possible should be used, the amount varying with different laths. About 3 or 4 lbs. of cow hair per barrel of cement is usually ample.

Thickness of Coat.—The minimum thickness of the scratch coat at any point shall not be less than $\frac{1}{4}$ in.

Application to Metal or Wood Lath.—The scratch coat shall be applied with a trowel and well pushed through the lath, so as to completely fill the space between the lath and the sheathing. Special care shall be taken to fill all voids around furring strips and where lath lap.

Application to Brick or Tile.—The scratch coat shall be forcibly and thoroughly troweled into the depressions of the previously saturated surface so as to make a firm bond. Care shall be taken to insure the complete filling by the mortar of all crevices and pores.

Curing.—In warm, dry weather, special precaution shall be taken to prevent rapid drying out of the scratch coat until a second coat is applied. Soon after placing and before the initial set has taken place, the surface of the scratch coat shall be roughened with a saw toothed paddle.

INTERMEDIATE COAT.

Proportions.—The intermediate coat shall contain one (1) part of Portland cement, three (3) parts of fine aggregate, and

one-fourth (¼) part dry hydrated lime. As little water as possible shall be used, that is, the mortar shall be applied as stiff as it can be worked to secure greater impermeability.

Application.—The intermediate coat shall be applied with a trowel over the scratch coat, previously thoroughly wetted, as soon as the latter has hardened sufficiently to be worked upon without injury. The thickness of this coat shall not be more than ⅜ in. and shall be well troweled to insure good contact with the scratch coat. Before the cement has attained its initial set, the surface shall be roughened with a saw toothed paddle. Adequate provision shall be made to prevent the mortar from rapidly drying out, either by sprinkling as soon as the surface will stand it or by hanging wet burlap or other suitable material over the wall to retard the drying.

Final Coat.

Proportions.—Proportions shall be the same as given for the intermediate coat.

Coloring Matter.—The proper amount of dry coloring matter to obtain the desired effect shall be thoroughly mixed with the dry ingredients of the final coat mortar, *i. e.*, before the addition of water. There shall be mixed at one time, if possible, sufficient dry material to provide all the mortar for the final coat of the work in hand, so as to ensure uniformity of color.

Application.—The finishing coat shall be applied to the thoroughly wetted under coat as soon as the latter has hardened sufficiently to receive it without injury. The thickness of the finishing coat shall not exceed ¼ in. when placed over the intermediate coat, or ⅜ in. when placed directly on the scratch coat. The finishing coat shall be pressed on hard with the trowel when applied, but as little troweling of the surface shall be done as is necessary to bring the surface to a smooth even finish.

Finish.

Smooth Troweled.—The finishing coat shall be troweled smooth with a metal trowel with as little rubbing as possible.

Stippled.—The finishing coat shall be troweled smooth with

a metal trowel with as little rubbing as possible and then shall be lightly patted with a brush of broom straw to give an even stippled surface.

Sand Floated.—The finishing coat after being brought to a smooth even surface shall be rubbed with a circular motion of a wood float with the addition of a little sand to slightly roughen the surface. This floating shall be done when the mortar has partially set.

Sand Sprayed.—After the finishing coat has been brought to an even surface it shall be sprayed by means of a wide long fibre brush—a whiskbroom does very well—dipped into a creamy mixture of equal parts of cement and sand, mixed fresh every 30 minutes and kept well stirred in the bucket by means of the whiskbroom or a paddle. This coating shall be thrown forcibly against the surface to be finished. This treatment shall be applied while the finishing coat is still moist and before it has attained its final set, *i. e.*, within 3 to 5 hours. To obtain lighter shades add hydrated lime of 5 to 15 per cent. of the volume of the cement.

Splatter Dash or Rough Cast.—After the finishing coat has been brought to a smooth even surface and before attaining final set, it shall be uniformly coated with a mixture of one (1) part cement and two (2) parts of sand thrown forcibly against it to produce a rough surface of uniform texture when viewed from a distance of 20 ft. Special care shall be taken to prevent the rapid drying out of this finish.

Pebble Dash.—After the finishing coat has been brought to a smooth even surface and before attaining initial set, clean round pebbles or other material as selected, not smaller than ¼ in. or larger than ¾ in., shall be thrown forcibly against the mortar so as to embed themselves in the fresh mortar. They shall be distributed uniformly over the surface of the final coat and may be pushed back into the mortar with a clean wood trowel, but no rubbing of the surface shall be done after the pebbles are embedded.

Exposed Aggregates.—The finishing coat shall be composed of an approved selected coarse sand, marble dust, granite dust, or other special material, in the proportions given for finishing coats and within 24 hours after being applied and troweled to an even surface, shall be scrubbed with a solution of one (1) part hydro-

chloric acid in four (4) parts of water by volume. After the aggregate particles have been uniformly exposed by scrubbing, the wall shall be thoroughly washed down by spraying with a hose to remove all traces of the acid.

Mortar Colors.—When it is required that any of the above finishes shall be made with colored mortar, not more than 6 per cent. of the weight of Portland cement shall be added to the mortar in the form of finely ground coloring matter.

A predetermined weight of color shall be added dry to each batch of dry fine aggregate before the cement is added. The color and fine aggregate shall be mixed together and then the cement and lime mixed in. The whole shall then be thoroughly mixed dry by shoveling from one pile to another through a ¼-in. mesh wire screen until the entire batch is of uniform color. Water shall then be added to bring the mortar to a proper plastering consistency.

WATERPROOFING.

When a special waterproofing compound is to be added to mortar for stucco work, it shall be added to all coats in strict accordance with the specifications of the manufacturer.

CURING.

Whatever finish is given the final coat, it shall be so protected from rapid drying out that the mortar shall have at least 4 days to cure before drying on the surface. The walls shall be frequently sprayed by means of a hose and in dry weather shall be protected by burlap or other suitable hangings which may be kept wet.

National Association of Cement Users.

PHILADELPHIA, PA.

STANDARD SPECIFICATIONS FOR SCRUBBED CONCRETE SURFACE

Adopted, September, 1911.

Forms.

Construction.—The face forms for all exposed surfaces shall be straight, smooth, evenly matched and watertight. They shall be so framed and fastened together as to be detachable and removable without prying against or jarring the concrete. Offsets at joints shall be dressed flush, and all openings at joints or corners, knot holes or other voids, shall be plugged flush with some stiff plastic substance, such as clay mixed with plaster Paris or sand.

Coating.—Before placing the concrete the whole surface of the face form shall be coated with petroleum or other water repellant to prevent adhesion of the concrete.

Placing Concrete.

Spading.—The concrete shall be thoroughly mixed, wet enough to flush, and be spaded against the face form, until the cement cream or grout follows and envelopes the spade. The aggregate of the concrete near the face shall then be pushed with the spade forward to the face, care being taken to obtain a uniform mixture and texture at the face.

Continuity.—The placing of concrete shall be continuous throughout definite stages, so that joints between different days' work appear at some feature line, or be made truly straight and level.

Removal of Forms.

The forms shall be carefully removed while the concrete is friable or pulverable, generally on the day after placing concrete, or later, depending on weather conditions and the setting quality of the cement used.

Scrubbing.

Original Surface.—Immediately on removal of forms the surface shall be scrubbed until the surface film is removed and the aggregate exposed to a uniform degree, and then the surface shall be rinsed off with water and kept moist for several days.

Patches.—If any void spaces appear or if spalls are broken off, the defects shall be patched with a similar mixture immediately after the scrubbing, using the hand or a wooden float—not a steel trowel—for applying and smoothing the patches. After the concrete has sufficiently set—say within from 5 to 24 hours—all patches must be scrubbed to the same texture as the general surface and be rinsed clean and kept moist for several days.

Tools.—For scrubbing the surface ordinary fiber brushes with a light stream of water from a hose or can, will be sufficient if the concrete is not too hard. If too hard, wire brushes followed by fiber brushes and rinsing, may be used.

Hard Spots.—If for any reason a portion of the face form can not or fails to be removed before the concrete has become too hard for scrubbing, the hard portion shall be treated with a sand blast or by tool-dressing to a texture matching the scrubbed portion. After tool-dressing, the surface shall be cleaned by washing with a 1 to 10 solution of muriatic acid and water, then thoroughly rinsing with water to remove all traces of the acid.

Granolithic Surface.

A granolithic surface shall be made by placing next to the face forms, in advance of the body concrete, a fine granolithic or facing concrete, thoroughly mixed and wet enough for flushing, composed of one (1) part Portland cement, one and one-half (1½) parts sand, and two and one-half (2½) parts pebbles, crushed granite or other approved stone as may be specified,

screened to pass a ½-in. and be retained on a ¼-in. screen. This granolithic must be maintained between the body concrete and the face form in a thickness of about one (1) in., care being taken that the body concrete is rammed into and perfectly unites with it. If iron or wooden mold boards are used to retain the granolithic concrete against the face form while placing the body concrete, care shall be taken that the board is not permitted to remain until initial set takes place, but it shall be frequently jarred and raised slightly at short intervals to prevent the formation of shuts and seams and air spaces between the granolithic and body concrete.

CORNERS.

In scrubbing, sand-blasting or tool-dressing, care shall always be taken to avoid roughening or blunting corners of the work and to maintain all edges sharp.

DISCUSSION.

Mr. E. J. Mehren.—I understood the report to read that the colorless surface treatments as a rule lost their efficacy in about two months, and after giving a list of those paints to state there were some that had demonstrated that they were permanent. **Mr. Mehren.**

Mr. L. C. Wason.—There are really here three classifications: (1) those which are so thin and clear as to make absolutely no discoloration of the surface; (2) those with considerable body to them, almost like an ordinary paint, which do somewhat change the color, but the change of color is a pleasing one; and (3) those of the tar or asphalt nature, which do change the color. **Mr. Wason.**

Those in the first classification, the very thin ones, did lose their virtue in a very few months, some almost immediately; in the second, they stood up for a very much longer period. Some almost indicate that they would for a sufficiently long period to consider them semi-permanent.

Mr. Rudolph J. Wig.—I would like to ask one or two questions; one with regard to separating the compounds made for the purpose of overcoming absorption. Also what method was used in testing, were they all tested in one way or were the compounds classified and tested according to their recommended purposes and uses? **Mr. Wig.**

Mr. Wason.—Where the manufacturers gave directions for the use of their compound those directions were carefully followed; and where they did not give directions a uniform method was used. Cubes about 6 ins. square were made by such process as to be porous, and with a hemispherical hollow in one face. Before they were treated they were tested with clear water and the length of time it took for the water to soak through them was carefully noted. Then after they had become thoroughly dried out again this hollow surface was given a coating of the material to be tested and was then filled with a measured weight of water—I think it was 100 cu. cm.—and a glass cover was placed over it to prevent evaporation. They were left in the laboratory for a while to watch **Mr. Wason.**

Mr. Wason. the percolation through them. After this had been done and this test had been completed the cubes were put on the roof of the building and exposed to the weather for a period of time of from two months to a year, and after they had been exposed to the weather for two months, water was put into them and the length of time it took to soak through was measured. This was usually a much shorter time than the time in the laboratory and was the basis of the statement that they did not stand the weather so well after exposure as they did before exposure. At longer intervals of time, 4, 6, 9 months and a year, the same experiment was repeated.

Mr. Wig. MR. WIG.—It seems to me that a very serious objection to this test is that a great many of the compounds are to be used as damp coatings. Some of them are made for the purpose of preventing the water from going from the exterior surface to the interior surface; others are made for the purpose of preventing absorption on the surface. To place the compound in a hollow and put water against the compound and leave it there permanently is not a fair test of the compound. You are not using it according to the purpose or recommendations of the manufacturer. The material is not made to be put in contact with water constantly. I think the value of such tests is very questionable.

It seems no attempt was made to separate these compounds, or to classify them, other than to test them according to one standard. I would like to ask how compounds which were to be used for preventing percolation were tested; that is, for waterproofing foundations, vats, tanks, or for such purposes.

Mr. Wason. MR. WASON.—The materials which were to be mixed into the mass of the concrete, classified here under the head of integral waterproofers, were mixed into the concrete according to directions of the manufacturers, exactly as they were given. The concrete was mixed with the dry process so that it would ordinarily be porous; and comparison was made between untreated blocks and those which had been treated, trying to use as far as possible a comparative method of testing for all, to see how far the various materials would fill the pores to render the block impervious to water.

Regarding the unfairness of allowing the water to stand

permanently in contact, the pressure, of course, was only that of perhaps 1 to 2 ins. head. This pressure is often exceeded by a strong wind combined with rain. The man who made the test did not see how it would be possible to spray material on a vertical surface, in the way rain would drive against a wall, and determine accurately the amount of absorption or the amount of material sprayed on it, so as to get comparable results. If you can devise some method of spraying or treating a vertical surface which would give accurate results I would like to know how to do it and try a series of experiments in that way. **Mr. Wason.**

Mr. Wig.—In regard to the integral compounds, I think it is a very serious objection to test them by placing the water in a hollow. Integral compounds—it is unfortunate that they have all been named waterproofing compounds, as they are not waterproofing compounds and they should not be classed as such—they are to prevent absorption and should be tested in that way. **Mr. Wig.**

We have made a great many tests and have devised a method to determine the value of the compounds for preventing absorption. Test pieces were made up which would absorb water, disc shape 2 ins. thick, 7 ins. diameter, and placed in a pan with a half inch of water. The water would be absorbed to the top surface, and by securing filter paper with flour paste to this top surface and painting the filter paper with phenol-phthalein all water absorbed to the upper surface was indicated by coloring the filter paper red. This certainly is a more fair and practical test of the compound. I do not think this test of placing the compound in water and subjecting it to even a very slight pressure is a fair test. It does not give to the concrete block or cement products manufacturer any idea of the value of the compound for overcoming absorption, as he is trying to overcome the dampness in the wall; and I think some other method than that employed in these tests ought to be obtained for testing the compound in a fair way and give the knowledge desired.

Mr. P. H. Bates.—I cannot agree with Mr. Wig at all. The compound may be and quite likely is going to be in contact with moisture all the time, and therefore any test that will have moisture or water continuously on the compound will be of more value as a test than the one in which water is not in contact with it all the time. **Mr. Bates.**

Mr. Bates. The matter of waterproofing or using these compounds depends upon their durability, and there is desired durability in contact with water, not in contact with dry cement at all. Now again, take for instance a compound like lime, using that as a waterproofing compound. If water percolates through that—especially a majority of our waters, which contain bicarbonate—the lime is going to be dissolved out, and for that reason the test designed for use only as permeability tests, which consists say of letting water flow through the specimen at the end of 7 or 28 days, does not have the value it should. To say that that specimen will allow so much water to flow through at the end of 28 days or 24 hours means very little; since if you should leave them constantly under pressure, you would find almost invariably, if you use pure water, the permeability is increasing constantly. You have there the actual contact of the water with the compound; and in the majority of cases this results in decomposing the compound and there must be something in the test to show that. Unless all the tests were carried on at one time I do not think they were comparable. If you put the specimens out on the roof say for 2 months during the summer, when we have very little moisture, they were not comparable at all with tests carried on in the spring and fall, when we have much of it.

Mr. Wig. MR. WIG.—From our tests we found that hydrated lime added to cement may increase the absorption, which would seem natural in that a very finely divided material is added. Hydrated lime, however, decreases the permeability decidedly. We found that by adding hydrated lime to a mortar and exposing it after hardening to a head of clear filtered water, the water in passing through the mortar would carry this hydrated lime and close up the pores, stopping the flow of water through the mortar entirely in a very short time.

We can take a sand mortar which is normally permeable and add clay or finely ground sand to it and expose to a head of clear water and it will become impermeable in a very short time. Using coarse aggregates increases permeability rather than decreases it, exposed to a head of clear water and increases quite rapidly, there being no fine particles to wash through and plug up the pores. In time it will enlarge the pores and disintegrate the mortar. Those

are our results, and I still maintain there are serious objections to Mr. Chapman's tests, which I believe were made on 2-in. cubes with a head of water of about ½ in., 100 g. in a hollow 2 ins. in diameter. I think this fact is a very serious objection to all the tests, and also the other objection of not being able to duplicate the tests. Another thing, the spreading rate ought to be taken into consideration in applying any paint coating. If this is not considered no two materials will be applied in the same way, nor is there any positive knowledge of the conditions or value of the tests. **Mr. Wig.**

Mr. H. S. Spackman.—The question arises in my mind of making the test specimen perfectly porous. For instance, using some solid matter like hydrated lime with conditions like a filter or sieve, if the openings were large enough the hydrated lime would fall through and have no effect. If they were small enough it would stop there and clog them and water could not get through. Now the question is whether in some of these compounds by making the concrete purposely more porous than ordinary concrete, whether you do not get an adverse decision against a waterproofing material which would not take place if used on normal concrete. Has that thought been considered at all in the tests? **Mr. Spackman.**

The President.—Apropos of that and as particularly germane to these remarks, just prior to coming to the convention, one of the book agents in Philadelphia who keeps me supplied with books, both modern and ancient, bearing on the subject of cement, sent up a book which was published about 1860, a little bit of a brochure of about a dozen pages; and among other things it had in there a letter that had been written by one of the big New York engineers; but I was amused in finding there, written away back in 1860, before concrete waterproofing had had its birth, attention called to the fact that impermeability was a great and desirable thing in either mortar or concrete, and it seemed that the stability of the structure would depend upon it; and he went on to say that as a result of his experiments he found that by getting a great density in the mortar or concrete, by selecting materials that went into it, it would not be necessary to use waterproofing at all; and he touched on hydrate lime as a medium of waterproofing. So you see the same story comes back to us and there is nothing new under the sun. **The President.**

The President. I have never been in sympathy with the tests made by Mr. Chapman. I have concluded that the means of testing those waterproofing compounds was a very crude measuring device. It gave very grossly incomparable tests. I think the question of waterproofing is covered largely by one of the conclusions of the committee that very little of them are of any value at all. And I think that while the exigencies of the present block or brick manufacturer may necessitate waterproofing, in the vast majority of cases you can so make your product as not to be necessary to use the material. Certainly as far as the massive reinforced concrete work is concerned, it is absolutely useless to attempt to use waterproofing compounds, for the reason that you can make the concrete in itself impervious, and it remains for the designing engineer to so distribute the shrinkage cracks resulting from temperature changes and otherwise to make them so limited that there is no continuity through the structure. And if that be the case then waterproofing is not necessary to the watertightness of the structure.

The report of the committee which we have here, of course, embodies the views of the men who compose that committee. I do not suppose it is proper that the Association should go on record as approving that, and I think the matter should only be discussed and perhaps published as information. I think that the essential features of waterproofing should be brought up, and I think that Mr. Wig has touched on some points that are essential.

There are two conditions in which waterproofing compounds are used: one in which the question of the absorption of the material is under consideration and the other in which the material is supposed to resist pressure; and under those conditions it probably can be effected by means of the reinforcing metal without the use of the compounds at all.

Mr. Wig. MR. WIG.—I would like to ask if there is any way of making a concrete product that will not be absorptive or will not discolor when exposed to moisture and if that is not one of the most serious questions confronting the cement products manufacturers? They want to get something that will prevent the face of the block from being discolored after a rainstorm of 3 or 4 days. Is there any way of obtaining this desired result? I believe from our tests

that in time the block will come to that condition if it is made properly; but in the early periods, up to say 2 or 3 months or a little longer, something is required to prevent this discoloration. I would like to ask whether there is any way of preventing it without using a foreign medium of some kind or if this is a condition which must be assumed as impossible to overcome? Mr. Wig.

The President.—The Chairman appreciates thoroughly the present commercial conditions of any such product as artificial stone, which perhaps possesses fair density. It was surprising when the committee took up the question of what the absorption of that stone should be, to find among all the artificial stone manufacturers but one to insist as high as 10 per cent. as the minimum absorption of that stone. Now I think it is manifest, with an absorption of that amount, that the chances are the soluble salts of the product will come out on the surface and discolor it and stain it. At the same time it is possible to make stone, and they do make stone abroad, that does not have such absorption; and while there may be a temporary stain on the structure where the absorption is low, I think it weathers off, and you have no decided stain after a time. Of course, if your product under normal conditions has as high as 10 or 12 or even 15 or 20 per cent. absorption, you must, of course, resort to something that will reduce that. But I say the use of such a material is a makeshift, and it is putting a premium on lack of skill in the manufacure of the product. The President.

Mr. Robert Havlik.—As I understand these tests, they were merely made for absorption and not permeability. Mr. Wig brought up the question of permeability and criticized the method used in these tests on that account, but I agree with Mr. Bates in criticizing the tests on account of the varying weather conditions. The amount of absorption would probably vary with the amount of moisture in the atmosphere. If there were but a little moisture in the atmosphere, as would be the case in dry weather, the absorption would probably be greater than on days when the atmosphere was very damp, and for that reason I think this is the main objection to the methods used in these tests. Mr. Havlik.

Mr. Wason.—Mr. Chapman, who made these tests, is not present to answer in detail. I have looked over the records in his Mr. Wason.

Mr. Wason. laboratory. It is my recollection, although I am not positive of it, that every one of the test pieces was not considered tested until it had been out in the air for at least a year, so that some time during the year it has been subjected to all the varying weather conditions. I am not sure whether they were all put out at one season of the year or not. I do not see, after they are a month or so old why there should be any great difference in final results if a month old specimen is put out in the fall and another one in the spring, if they all go through the same climatic conditions in the course of twelve months.

Mr. Boynton. MR. C. W. BOYNTON.—Mr. Wig is undoubtedly correct in suggesting that the discoloration or efflorescence of porous blocks is the thing that makes the block manufacturer seek some waterproofing material. I am also convinced that the discoloration will vary with the porosity, and if a block is made dense, as it should be, the danger from this source will be reduced to a minimum.

We are here to promote the proper use of cement and concrete and should be careful not to adopt rules or regulations which will popularize a practice which will result in cement products being less satisfactory than competitive materials. As cement block, for instance, can be made practically impervious and strong by the use of proper materials and proper methods, this Association should not countenance any practice which will not produce the very best results. If a concrete block is made of proper materials and by proper methods, it will not require waterproofing. In my opinion waterproofing is resorted to in most cases because the manufacturer is making an inferior product, which would be very apparent if he did not treat his block in some way to make it repel moisture. This treatment, however, may only be temporary, and the poor quality of the block is apt to develop later to the disgust of the purchaser or builder.

Mr. Gill. MR. J. ROBERT GILL.—I would like to get some expression on the question of whether or not the use of a dry facing, that comparatively dry facing which gives a granular texture and which is the common cement product finish, is to be entirely discouraged. Having in mind my experience with the dry facing I mean a consistency that is wet enough to enable it to be compacted, so as to get a dense, hard finish. It gives a certain granular texture,

but is not subject to crazing. Now, of course, you can obtain a dense facing by using the cement wet enough to run, but such practice will tend to cause crazing, except, of course, when methods of brushing or rubbing or something of that kind are resorted to. In other words, I understand the trend of the discussion to mean that we are to resort entirely to rubbing or scouring faces or like treatment subsequent to molding, if we are to get any kind of finishes other than the ordinary concrete surface. Mr. Gill.

Previous speakers seem to indicate that the field for concrete blocks should be limited only to those methods of finish such as brushing or scrubbing or some other plan that would take off the film of cement; because in my experience if you use the facing wet to get the density it is inevitably subject to crazing unless the surface is given subsequent treatment.

The President.—I think the trouble with the cement block business is the fact that it is a molding proposition, and being a molding proposition necessitates taking the product out as rapidly as possible; the very nature of it demands that the mixture shall be dry, because with water there is more or less apt to be a breaking of the face. The President.

There is a tendency in the manufacture of blocks to resort to a facing material, perhaps a white or other facing, to make the block attractive, and the use of some facing mold that will produce regular face effects. Now as long as neat cement covers the surface the same effect will be had as on the face of the mold. I believe that the block industry of the future is one in which the skin will be removed either by cutting or tooling or some other way of exposing the aggregate to remove the surface. If this is done it seems that there is no reason why the concrete should not be mixed with little water; the very act of mixing concrete is one of the most important things to be studied. It does not necessarily follow that to get wet concrete it must be sloppy, so it will run out of the molds. If the material is properly graded and proportioned for density and enough water is used so there is a thorough mixing, a viscous, heavy mass can be obtained with just as much density as with little mixing and a whole lot of water.

With the proper density of concrete, waterproofing compounds are not necessary. The absorption of the stone is reduced to a

The President. point that the discoloration, that may come from any slight absorption, is unobjectionable. There should be studied the methods of making blocks of stone that are to be surfaced in some way such as Mr. Watson has told us at one of the previous meetings, how he takes his blocks and runs a series of carborundum bricks over them so as to tool and take off the skin or surface. As to the facing, a dry facing can be run in, and take off that dry material, and if it is properly done it will enable the material to settle.

Mr. Havlik. Mr. Havlik.—I have made many experiments in trying to produce waterproof block without the use of waterproofing. I was a firm believer 3 or 4 years ago in the possibility of making waterproof concrete, especially blocks, without the use of any waterproofing compound. I still believe that in monolithic work it is possible for all practical purposes with the use of graded aggregates. But the average concrete block manufacturer has not the facilities for grading his aggregate, and he may not be equal to grading it, that is, he may not have the technical knowledge required for carrying on the necessary experiments, consequently he must use whatever material is at hand or whatever he can get for a reasonable price.

I have made tests of ordinary sand, have used marble dust, and marble dust, white sand and white cement. I have made a facing so wet that when the block was removed from the machine a film of water could be seen on the face. I think all of you will agree that this is pretty wet, but I never made one block under that condition that was absolutely waterproof. If I put water on it it would go through the block. It did not go through rapidly, but this is the point, the water spreads on the surface of the block and is slowly absorbed, and the big problem the block manufacturer has to contend with is to supply the contractor, or the man who is building a home, with a material that absorbs very little or no water.

The concrete block is something new. Ten years from now we can make a concrete block with as little absorption as natural stone and have it used in place of same without any great difficulty, but to-day the man who wants a really fine house and will consider blocks, will not consider them unless you can guarantee that they will be absolutely waterproof or nearly so; that is, that they will not take more than about one-half of one per cent. of water.

Mr. Boynton said he believed a reasonable man would accept a block that had as little absorption as stone. We can produce that kind of a block very readily, but in trying to secure a few contracts like that we were unable to secure them unless we guaranteed that the blocks would not take moisture, or would take far less than stone. So on that account I think it is a very serious problem to the concrete block manufacturers throughout the country to secure a waterproofing that will produce these results. In years to come we may reach the point where the builder will accept a block that is only as good as stone, but to-day on account of the fact that the concrete block is practically new, it is absolutely essential for the block manufacturer to make a product that is absolutely waterproof or very nearly so. We must do this to secure the approval of the average architect. Mr. Havlik.

We have a big problem on our hands in South Bend. The architects there are not greatly in favor of blocks. In many ways we cannot blame them, for they have seen only the ordinary gray blocks until the last few years. But these same architects when shown a real good block that does not absorb any water have specified same in new buildings, and we have secured a contract for a two-story Interurban Station at Elkhart, Indiana, simply because we could give the owners a block so waterproof that when water was placed on the surface it did not disappear until it was evaporated, and none of the water was absorbed. This is a test that anyone can make very easily. If the water spreads in the least on the face of the block it is being absorbed by same. If it stands out in a film it is not being absorbed.

Mr. Wig.—I would not like it to be understood that I advocate the use of waterproofing compounds. In all our tests in making up all these mortars we had to prepare a special mortar. As the President has said, we had difficulty in getting water through any normal mortar. It would not go through it. But at the same time we had trouble with this absorption, this discoloring of the surface. I might say that I am very skeptical of all waterproofing compounds that are on the market at the present time. Mr. Wig.

The President.—I think the surface Mr. Havlik speaks of where he has half of one per cent. absorption in a block, can be obtained more properly with cravenette, as our friend explained The President.

The President. last evening, rather than with a waterproofing compound. I think that is going to the extreme. Granite has an absorption, but I do not think it is that we are trying to obtain. Reverting to the convention in Indianapolis in 1905, a glass of water poured on the best block exhibited would go into it as fast as poured. It is evident that in seven years we have made progress, because if you go down to the blocks that are exhibited this year some of them will shed water, but most of them will not be waterproof.

Now I have seen in Germany blocks made in a similar manner and on which water would stand on the surface, and there was no waterproofing compound of any kind put in it. The reason the block had such resistance to the absorption of water was due to the fact that they used elbow grease in mixing, to work the air out of the mixture on getting it into the molds, rather than resorting to waterproofing compounds to do the same thing.

TOPICAL DISCUSSION ON TREATMENT OF CONCRETE SURFACES.

Mr. Ballinger.

Mr. Walter F. Ballinger.—We are only in the infancy of the development of the architectural effect of concrete for exterior surfaces. Not only does the architect and engineer not know a great deal about it, but we find the artisans who should be skilled in doing that kind of work are not skilled to any great extent.

Of course, the aim is to treat concrete as concrete and not to imitate anything else. The scrubbed effect is handled practicably for surfaces where the forms can be quickly removed. For the soffits of arches and so forth, however, it is not practicable and tooling is probably the next best thing to eliminate skin effect or water surface of the concrete which develops hair cracks and shows up unsightly in time. We are under the impression that the cracks are there just the same, but they are not seen in the scrubbed surfaces, and are probably not quite as large because the finer particles of cement get to the surface and shrink more and therefore crack more than the parts underneath.

There is one development of concrete work I think there are great possibilities in, and that would be the artistic treatment of concrete blocks. We have used concrete blocks in a few instances and in every case regretted it, because the effects desired did not materialize. If persons would go into the concrete block business, as they go into any other manufacturing business, with an adequate plant to do the work and with a knowledge of concrete, how to treat it, and a knowledge of the views of the architect on the subject, I believe a very successful business could be established. Cement blocks made with careful selection of the aggregates and different kinds of stone would be successful. The treatment should be varied, some blocks with granite surfaces and some with other kinds of materials for the aggregate. Broken glass on the surface may present a very good effect, though I have never seen it. Such blocks, after being cured in a steam room, might be put in a polishing machine to obtain an effect similar to terrazzo. Concrete might be used in that way for exterior surfaces and give as pleasing an effect as very high price stone and at much less cost.

Mr. Quimby.

MR. HENRY H. QUIMBY.—Mr. Ballinger's impression that although they are not seen, hair cracks do exist in scrubbed and tooled surfaces as well as in surfaces that have not been treated or are treated in the old-fashioned way of painting with neat cement, may have some substantial basis, but I know of none. I have yet to see the first evidence of them. If they are so minute as to be invisible they cannot be serious, and there is no denying that the ones that are visible in the smooth surfaces are decidedly objectionable.

The scrubbing produces very much the same surface as deep sand blasting, but has the advantages of lower cost and more easy removal of fins, offsets and other blemishes caused by imperfections in the forms. Sand blasting, of course, can be done at any time while scrubbing requires the removal of the forms say within twenty-four hours of casting. Tool dressing is a very satisfactory method and if done with an edge tool or bush points and then cleaned with acid, it will fairly match the scrubbing in appearance.

The matter of appearance in any work is in nearly all cases a question of taste and taste is largely dependent upon temperament, association and thought, and is often a development. One of the earliest if not the very first of the journals devoted wholly to the cement interests in its early days regarded the backwardness of architects in using concrete as due to its unpleasant appearance when either untreated or treated in the then prevailing methods, and it hailed the development of the scrubbing process as removing the greatest obstacle to the use of concrete for exteriors of buildings. Recently it contained illustrations of a dwelling house constructed of concrete entirely untreated and showing all over its wall surfaces, inside and outside, the impressions of the form boards, and the effect is claimed to be artistic.

I feel that in designing structures that are to be in the eye of the public they should be treated with a view of making them pleasing to that eye—that the miscellaneous taste should not be ridden over rough shod with an ultra artistic impression that may prove very fleeting and cannot be really educative.

Mr. Trego.

MR. EDWARD A. TREGO.—I think the scrubbed surface devised by Mr. Quimby is the best I have seen in the way of a treated surface, but I am quite convinced that concrete presents an excellent surface just as it appears when the forms are removed and that

eventually this will be accepted as satisfactory once we become accustomed to it. Mr. Trego.

Mr. George Prince.—I am inclined to agree with Mr. Trego that we are getting back to the point of accepting concrete surfaces very much as they appear when the forms are taken off. I believe that the beauty of a building or a structure, whatever it may be, depends largely on its design. Mr. Prince.

Mr. Quimby.—Referring to the board marks left upon the concrete as the forms were taken off which Mr. Trego seems to regard as attractive and artistic, I am reminded of the question of a visitor to an arch bridge, looking up at the soffit—there had been no finish on it—he asked: "Why did you leave the lagging on there?" Mr. Quimby.

An untreated surface creates an impression in the minds of some people that the boards are actually there, because the grain of the wood is reproduced in the surface of the concrete. I am not ready yet to abandon the practice of treating the surface of concrete to make it attractive.

The President.—A case where a concrete surface was used just as the forms left it is at the Mission Inn, Riverside, Cal. The old inn has been replaced by a very large hotel, and one of the wings used as a large lounging or ball room, is built in the mission type. The beams, the walls and all the parts, even the fireplace, are of rough concrete. The form marks appear there and it does not appeal as being unsightly or ugly, but impresses one as a thing of great beauty. The contrast with the redwood finish, rough boards on the surface of rough concrete, harmonizes with the scheme of decoration and makes the interior a thing that instantly impresses any one, whether of artistic temperament or a layman. I visited the inn with a party of gentlemen, of whom I do not think there was but one who had any claims to a knowledge of art, and we were unanimous in endorsing the beauty of the room. The President.

It seems to me what after all is sought in any scheme of decoration is to erect a structure pleasing to the people and beautiful. I appreciate the fact that we can cultivate this particularly by scrubbed surfaces and exposed aggregates and perhaps by tooling and faience and other forms, but this does not appeal to the people who must be educated to appreciate that style of art.

The fireplace itself had no molding. It has just simple plain, straight lines, substantial, rugged, and in the crown of the archway

The President. covering the cloisters on either side, the board marks of the forms were very evident. There were ridges perhaps an eighth or a quarter of an inch wide, where the boards did not match properly. It gave a very rough, unfinished appearance to the surface, but more an appearance that was pleasing to the eye. Of course, that was an interior exposure and it was in a climate where, perhaps, hair crazing and checking is not likely to occur. It seems to me the most objectionable feature of concrete is the smooth surface, where the cement has been brought to the surface in the process of pouring the concrete and produces cracks on exposure to the air; any process that will tend to eliminate the skin of neat cement and give the surface some life is of advantage. The method of sand blasting, of scrubbing and acid treatment, and even using a brick of carborundum or something to rub off the surface and bring the particles into relief, which gives life to the surface, seem to me a desirable thing. Of course, the cost of the treatment is an element which depends entirely on the conditions.

Mr. Wight. MR. FRANK C. WIGHT.—In considering the use of concrete in its natural form, there should not be lost sight of the fact that the architectural beauty of a building depends on two things: the grace of the line and the beauty of the texture. Now it is quite impossible, with the classic line that everybody has been using in concrete structures, to attempt to combine with that the common board texture of concrete. If this board texture is to be used and is to be beautiful, there has to be developed with it a different style of architecture for concrete and that style of architecture will be particularly difficult to apply to anything except buildings. The ordinary arch bridge or even the girder bridge as built in concrete to-day is a holdover from the stone design, and in its lines it is nothing but a stone arch; so that the surfaces of these stone arches have to be made and the concrete arches built in the style of stone arches and in conformity with our ideas of what stone should be.

While board surfaces may be applicable to interior decoration or to small or large houses, where the architect has the foresight to develop this new line, still in the majority of cases, large structures where concrete is being used, is bound to be used, common board wood mouse colored surfaces will never agree with the architectural lines being used.

Mr. Oliver Randolph Parry.—From the architect's view- Mr. Parry.
point, the matter of finish is largely decided by the particular class and character of each building. For instance, if you have an office, bank or other large building, where Indiana limestone or granite with a tooled surface naturally suggests itself, then you want a tooled finish and similar tone to the stone. On the other hand, if you are dealing with a house or similar building where the design be, say, English half timber, Dutch Colonial, or other of the styles one sees so extensively employed in this class of work, then you want most any finish other than of a tooled face, and naturally, will decide upon rough, smooth, pebble, inserted tile, color, etc., finish from the surroundings and the building itself.

In the case of a suburban residence, the owner desired a smooth sand finish and he procured it, but not in the accepted manner, as naught but smooth soft pine boards with handles nailed thereto were used. After the former were taken down, and whilst the concrete was still damp—the forms were removed quickly—the surface was freely rubbed with the boards until a slight effervescence, or froth, appeared. The walls were allowed to set without further treatment, although they were washed off by a slight rainstorm shortly thereafter.

In another case, a summer residence, the aggregate was extremely hard to work, and in many ways acted peculiarly. Immediately in front of the river was a beach with sand pebbles, and it was here that the materials were procured. Carborundum stones of various grades were used on the exterior and interior of these walls, it being found that at the stage when rubbing ordinarily would be the most advisable, the aggregate would not remain in the concrete, while, on the other hand, if allowed to set long, it meant that they would come to the surface and require much rubbing. The problem was solved by finding the exact time in which to perform the work, which was done by careful mechanics, and upon the rougher board marks being removed, a brushing (not coating) of water, with a little cement and fine beach sand, completed the finish. The white sand used for this purpose left a fairly white natural surface.

It would seem to me that the building itself must determine the finish and character of work. As an example of this, I will call attention to one of exhibits. No doubt there has been noticed on

Mr. Parry. top of the columns, or rather, figured pilasters of the pergola, weathered rafters, that is, what would appear to be wooden boards, but in reality they are cast of concrete. Now, if it is possible to procure the effect of a rough board so nearly that it fools most every one who looks at it, it must well imitate the other material, which brings us to the point that it would not be as cheerful to the artistic senses to see a plain, smooth concrete slab, perhaps of that uncheerful, cold gray, for this purpose, as it would be the natural board itself. The concrete, however, has demonstrated in this case that it may be used in place of the wood, and to equal effect, thus performing the function of a good imitator.

It would seem to me that concrete is the best material in general in building construction, for not only may you use concrete in concrete design, but you may use it in almost any other design for most any class or character of building; for instance, in English half timber, it takes the place of stucco, and in the colonial stone plastered house it does the same thing, and so one may go through the whole line.

As a further example in finish, a bank in the Georgian period, where red brick and Indiana limestone trimmings were a requisite, I desired to have of monolithic construction, but this was overruled, and concrete block particularly desired. With a vision of an opportunity to demonstrate to my own satisfaction that it is possible to use these blocks advantageously for the base, quoins, trimmings, etc., I interviewed the nearest manufacturer of blocks, taking a chip of Indiana limestone along, and carefully explained just what I was anxious to procure in the finished result. This bank has demonstrated to the speaker's mind that it is not only possible to match Indiana limestone for color and texture, and surface where desired, but that it may be accomplished at considerably less cost than would be the use of the stone itself, beside being more substantial.

Still another case may be cited of a dairy barn where the walls were left just as they were when the forms were removed, showing the ridges and similar markings, the result being an honest impression of the material thought sufficiently well to be employed in the building, and, in the speaker's opinion, more artistic than it would have been had treatment taken place.

Now, if we were dealing with a bungalow or smaller building,

on a country estate, where substantiability is desired in construction, and wood in effect, it would be highly possible to procure the same were forms to be made of rough hemlock left out to the weather, or so treated as to raise the texture and grain. By this process you will procure the same lines as if clapboards were used, and it is even possible to apply triangular pieces in such a way that the butt of the clapboards will likewise show. Going further, if you desire to treat this with stain, you can procure any color effect possible with wood. This will give an artistic finish which will wear and be substantial. You may even please the client who demands a great big stone fireplace, which must be on the outside, running all the way up through the roof. By the use of concrete, with filling in on the outer surface of the forms of the stone, this is possible, as it would be were the requirements to be of brick. **Mr. Parry.**

Concrete may be used nearly anywhere and for nearly any finish, the color effect being possible not alone through the use of pigments in the concrete itself, but by possibly the easier method of washes on the outside.

If you have a colonial house of a class requiring a smooth sand finish, that will be the finish which you will want for that particular house; while, on the other hand, you may have the same style of architecture where it is desirable to have a rougher or "pulled trowel" finish, which, likewise, may be closely imitated.

On the other hand, you would not want to go down Broadway and see a large office building or bank with a wood grain on the pilasters, yet this might be very pleasant in a bungalow or smaller building. Regarding the finish of concrete for interior work, the possibilities as shown in the exhibit are so wonderful that well may the average practitioner open his eyes and become wedded to this construction.

Mr. Trego.—I would like to call attention to the new Pennsylvania Station in New York City. It looks almost like concrete, not only like concrete, but if it had been a concrete job I do not think it would have been accepted without plastering owing to all the cracks and depressions in the Travertine stone. It looks exactly like concrete with the board marks in it. I think it has a very close resemblance, and is certainly a very large and dignified building. **Mr. Trego.**

The President.—In the lobby of the Pennsylvania Station, in the main gallery that leads to the general waiting room, the wall is **The President.**

The President. composed the first four feet, I think, of natural Travertine stone. Above that it is an imitation of this stone, made of concrete; and often the workmen on the building hesitate and look pretty closely before they can tell where the natural stone ends and the artificial stone begins.

In one case the owners of a building did not wish the surface treated after they had seen the unfinished surface. The weather is liable to do for a surface of concrete what we do artificially, take off the film. I have seen some concrete work that was old, which had been dressed by the weather, and it was really very hard to determine whether it was artificial or natural stone, because the action of the elements takes off the film of the concrete and exposes the aggregate.

Mr. Atwood. MR. P. H. ATWOOD.—Can an artificial finish, a stucco or pebble dash be placed on a smooth block wall without first applying a wire coating a mesh?

Mr. Holt. MR. THOMAS S. HOLT.—I have had some experience in stuccoing cement blocks. After some difficulty with dampness, I decided to make the blocks with a plain finish, such as the hollow tile, and to stucco the building on the outside. There is no trouble at all in placing stucco right on cement blocks which are made by the semi-dry process and are somewhat porous. To remove the plastering would necessitate taking away part of the wall. While there might be difficulty in stuccoing on a clay brick wall so as to always insure a good bond, I find there is no difficulty whatever in plastering on the cement block. By giving it a stucco finish we have a very attractive building which looks very much like a reinforced concrete building. I just completed such a building about a month ago. It is my favorite type at present, and very economical in construction. Moreover, there is a possibility of reinforcing the building during construction and it can then receive the desired stucco finish at comparatively small cost, securing a very attractive appearance.

Mr. Richardson. MR. A. E. RICHARDSON.—Last year we stuccoed a house direct on the blocks, and it went through the winter in fine shape and has a very good appearance now. This year we built a garage and plastered direct on the blocks. There is a good bond all over except on some blocks that were very wet. I have noticed some hollow spots on the building where the blocks were especially dense, and this is caused to a great extent by the use of old lime mortar mixed with cement mortar by the masons. There are only one or two

places where it is hollow, otherwise there is a perfect bond. The blocks we use are made by a medium wet process and where we did not get a bond they were very wet indeed; that is, no water would be absorbed. **Mr. Richardson.**

Mr. Holt.—It is necessary to first put on a very thin coat to prevent lagging or sagging of the material. If one heavy cement coat is placed when the material is a little too wet, instead of being held in place by the bond the outer particles will slide, producing a hollow space and destroying the bond. I always place a skim coat and follow it with a second coat. There is no difficulty whatever in making it stick and obtaining a good bond. We use a small percentage of lime in the mortar for all coats to make it plastic and easily troweled. Cement mortar is very short of grain and very difficult to work. We did not experience trouble in putting 25 per cent. of lime in the mortar and it worked very nicely, giving us a perfectly waterproof plaster. That percentage of lime, of course, made it trowel very nicely and gave better results than obtainable without the lime and there is no sign of hair cracking in the surface. **Mr. Holt.**

Then on the interior of the building—a factory building—we plastered directly on the wall. Instead of 25 per cent. lime we used 75 per cent. lime and 25 per cent. cement in the mortar, which gave very good results.

Mr. W. M. Newton.—With a comparatively dry block there is considerable absorption when the mortar is applied and it is an apparently easy matter to put the plaster on. A block made very dense and wet, that is, such as ordinary reinforced or monolithic concrete, is very difficult to plaster. It takes a good deal more skill to apply a coat to a wall that is dense than to a very dry block. **Mr. Newton.**

Mr. Prince.—Cement blocks have been made from wet material in wooden molds, and scrubbing the surface to expose the aggregate. It looks very nice, giving a decidedly different appearance from any concrete block work that I have even seen before. The blocks were cast in wooden molds and as soon as the concrete had set sufficiently one side of the mold was turned down and the surface scrubbed so as to expose the aggregate. **Mr. Prince.**

Mr. J. Robert Gill.—Last summer we were confronted with the question of making blocks of a little different shape from those made to advantage in a machine; that is, a block about 4 ins. thick **Mr. Gill.**

Mr. Gill. and 4½ ins. high; and we came to the conclusion that the simplest way was to make the blocks in wooden forms. A wooden frame was devised that made a panel of fourteen blocks 24 ins. long, or could be divided up so as to get different lengths than 24 ins.; that is, it would make two blocks such as 8 and 12 ins. long instead of one block 24 ins. long. In fact, all length with face 4½ ins. wide could be made. Over the pallet was first placed a layer of oiled building paper, the pallet being on a flat car and moved around as the process progressed. A crew of three men mixed cement and crushed cobblestones from ¾ in. down to fine, but no dust; this was poured real wet into the molds. Having poured in this mixture it was followed with a little pure cement, very thin, so as to thoroughly fill in between every particle of stone. Then this car was moved to the mixer and filled in with what might be called a puddle mix of concrete, rather wet. Then when it was thoroughly filled and struck off, the car was placed in the steam room for approximately 3 hours rather hot steaming; then the car was taken out and the pallet containing the 14 blocks in the panel slid off on to a skidway, the clamps put on again and turned it up on edge. The oiled paper released readily and left the surface somewhat soft, so much so that it could be scraped off with a thumb nail. With a scrubbing brush and a stream of water from a hose the cement was removed rather deeply, that is to say, so as to bring out the granite in quite strong relief. Then the panel was turned down on the pallet again and placed in the steam another 2 or 3 hours; then brought out, the forms removed and the blocks placed face up on another car, and left on that car ready for the final treatment the next day or the second day. This consisted of a solution of acid and water put on with a brush and thoroughly rinsed. Nothing but the crushed cobblestones, which the city used for street paving and the county for country roads, gave a very pleasing effect, both as to color and light and shadow effect. In fact, the process was very successful all the way through.

Mr. Quimby. MR. QUIMBY.—The improved methods of making concrete blocks have advanced their use very much. The first ones apparently were all made wet and dense and the surfaces not treated, and buildings made of those are usually unsightly. With the more porous blocks, the treated kind, much better results are obtained. On a recent trip through the city of Havana, in reply to a query

as to what kind of stone was used in a certain very large and fine building, the guide stated they were cement blocks. The blocks were smooth, evidently made with uniform size coarse sand and apparently untreated. The mixture had apparently been dry, for there was no segregation against the forms. The surface looks just like sandstone, and there was no effiorescence. That may be credited, probably, to the weather of the climate, but it was an exact imitation of stone. Now if concrete blocks can be made with such a satisfactory surface why should stucco be applied? Mr. Quimby.

Mr. Fred G. Webber.—I believe the practice of applying stucco to concrete surfaces of one character or another, should be discouraged. This statement may appear a little strong, but, on serious consideration, I think that you will agree with me. Most any man who can handle a hawke and trowel can go to a job and apply mortar to the surface of the work; he can make a first-class appearing job of it; but what happens after he leaves the work, the next year or the year after? As a general rule the elements have to be considered—or rather the elements do not consider who applied the mortar or upon whose building—but they go to work and break off a piece here and a piece there. Mr. Webber.

Now it is quite possible to treat concrete and obtain a presentable surface by either of several methods. In the matter of leaving the surface, as it comes from the forms, show the markings of the wood, I believe that concrete should stand for concrete and not for something else. It might be said that leaving the surface as it comes from the forms, showing the grain of the wood and the joints, that it becomes more or less a wood structure.

As to the several methods of treatment mentioned, the scrubbing of surfaces by brush and water or etching with acids require considerable skill and attention, and when attempted should be undertaken with the idea in view that close supervision is absolutely necessary to turn out first class work. Scrubbing can be done commercially only within a very few hours after the concrete has been poured; and in case of a day or two of storms directly after pouring, the contractor is in difficulty when he attempts to reveal the aggregate by the brush method. Then comes the question of doing the work with acids, and if some of the work has previously been executed by the brush method, I doubt very much whether it could be matched in texture.

Mr. Webber. When a proper aggregate has been used, I am greatly in favor of the tooling process, and believe it is coming into favor more and more every day. In a very short time contractors in general will be able to execute work in that manner as cheaply as they can by brushing, without the many drawbacks that go with brushing or acid etching.

Mr. Quimby. MR. QUIMBY.—It is possible, where a contractor is caught in bad weather and cannot remove the forms from a part of the work in time for scrubbing, to match the rest of the work. It is not necessary to etch with acid which is a troublesome and expensive process, particularly on monolithic work. It is, perhaps, necessary where part of a job has been scrubbed to wash the tooled surface with a weak solution of acid, not stronger than 1 to 6 or 1 to 10, to clean the cement smear off the aggregate. Then a very good match of the washed surface results and the same effect is obtained because of the same exposure of the same aggregate. However, exposure to the weather will in time do the same work as the acid.

In reference to tooling, the two most essential things for a finish of concrete are to have the edges sharp and the texture of the surface uniform. Round edges always look weak. It matters not so much what the aggregate is; any aggregate that is good enough for a surface is good enough for looks, as a general principle. That is, it will look nice only if it is uniform.

Mr. Newton. MR. NEWTON.—Plaster surfaces for exterior work may be practicable, but the skill of our mechanics must first be considerably developed.

A point which should not be overlooked is that in the finish of a surface the weather and time will bring out the aggregate. It seems that if the money represented by the cost of tooling and etching or sand blasting were put into the forms, producing panel, molded or good shadow effects in a building, and allow the texture to result from natural causes, that perhaps in the long run we would get more for our money. This does not refer to the high grade hotel in the center of a city where immediate effect is necessary, but to the factory building in a large city which is desired to look fairly good at once, but the initial cost must be kept down. The money should be spent on the forms rather than on the etching and the finish of the surface, so as to obtain as perfect a surface as

possible with the forms and panel effects and then depend on Nature for bringing out the texture of the surface. Mr. Newton.

Mr. Holt.—There is another way of treating surfaces and that is the use of the rubbing brick for removing the form marks. Some specifications—the State Road Commission—require the rubbing down and removing of all form marks. A rubbing brick was used and the wall was wet with a brush and then worked with the brick. This produced lather which was floated up on the surface and in that way the sharp form marks were removed and leveled down pretty nicely. In my observation there was some objection to the use of the brick and I would like to obtain some expression from others who have used the brick on reinforced concrete surfaces. When the concrete is rubbed a kind of limy substance comes to the surface, and there is a question as to the action of the weather on that. I have noticed in some work so treated a tendency to hair cracking, which did not appear on work left in its original state on removal of the forms. In my experience I have never had hair cracking, excepting where something had been done to the surface after the forms were removed either in the way of rubbing or plastering. Mr. Holt.

Mr. Denman.—With the use of a carborundum brick, the effect will usually be that of a very mottled surface. This can be removed to a very large extent if after using the brick, the surface is rubbed again with a brush and clean water, removing the superfluity of cement brought to the surface. Mr. Denman.

In regard to tooling, there has been a question as to the use of skilled labor. To tool or treat in some way a very large spandrel wall surface, a compressor capable of handling four tools was installed. It was thought necessary to get high class experienced men for running the pneumatic tools, who were paid 50 cts. an hour. They worked so slowly that two more pipes and tools were installed and a couple of Italian laborers from the concrete gang placed at the tools, and at about one-half the wages tooled about twice the surface area the others did, and equally well, after simply showing them how. Consequently the skilled labor was taken off and two more Italians set at the work. With a foreman and a superintendent who understand the treatment and can show how it is to be done, skilled labor is not needed for treating surfaces of concrete generally. The ordinary laborer can either brush, which is

Mr. Denman. certainly far the cheapest, or use the pneumatic tool or wire brush afterwards just as well as any skilled laborer.

Mr. Webber. MR. WEBBER.—The average mechanic on work of that character does just enough to hold the job; while the other fellow, who is trying to rise in the world and get a few cents more a day, will turn out from two to three, and I have had them turn out four times as much work as the skilled mechanic and at less than half the wages. I am not an advocate of using skilled mechanics upon the tooling of concrete in general. You can select from an ordinary gang a crew sufficient to tool any job. I have 60,000 or 70,000 sq. ft. to do and the only mechanic used will be the man to sharpen the tools.

Mr. Denman. MR. DENMAN.—I want to ask what has been the general experience as to the best tool for simply giving a uniform surface, whether it is the bush hammer, and if so how many blades should be in the tool, or what tool has given the best result. I have used a lot of them and think the tool about 2 ins. sq., with about 16 star points on it, will work the fastest and give a uniform surface. I do not mean by a surface in which all the aggregates are exposed, but one in which all the superficial film of cement and the board marks are removed.

Mr. Webber. MR. WEBBER.—I think it a mistake to use a bush hammer on concrete surfaces unless the price is very good. It, in the first place, gives a surface that is very fine and in the second place, which is most important, it is very expensive to the contractor as the dust clogs the hammers, filling the spaces between the blades. A strong appearing surface is obtained by the use of the tool the last speaker mentioned, that is, the head is square with either 4, 9 or 16 points. They will remove the surface about as rapidly as any tool leaving a good appearing job and they can be sharpened very easily—simply the temper drawn and sharpened with a file, or if the tool is too far gone there is a special tool to reface the cutting tool.

Another cutting tool that has given a great deal of satisfaction is a sort of saw-toothed tool with a half a dozen points on its surface. This cuts rapidly and well, and a great variation can be given the surface by the angle at which the tool is held.

Mr. Quimby. MR. QUIMBY.—I have taken occasion to look carefully into the methods of dressing tools, the advantages of different tools, and I have about come to the conclusion that there is very little difference in the amount of work that can be done with the different

tools. I have used bush hammers with points from 1 in. square with 9 points to 2½ ins. square with 16 points. I have seen concrete dressed with the crandall or bush hammer that has 4, 5 or 6 sharp edges grouped together, and with a single edged ax all by hand, and am not prepared to say that one does more work than another. The only tools that can be said to call for any skill in the man who handles them are the ax edge and the crandall hammer, because they require that the cuts all be made in the same direction. Mr. Quimby.

For the pneumatic hammer a single chisel edge is probably the most advantageous because most easily kept sharp. Several speakers have stated that skilled mechanics, so called, stone cutters, do no more work in dressing a surface of concrete than ordinary laborers do. The laborers accomplish as much as the mechanics, but I cannot say that I have ever seen them do three or four times as much. I have observed that about 10 sq. ft. per hour is the output of the man with the tool in his hands, merely removing the surface film and at the same time cutting down any little offsets that there may be. Twice as much or 20 ft. per hour is perhaps as much as can be properly done with a pneumatic tool.

TUFA CONCRETE.

By J. B. Lippincott.*

The Los Angeles aqueduct is designed to be 215 miles in length, extending from Owens River on the eastern face of the Sierra Nevada, to the city of Los Angeles. This aqueduct will consist of 41 miles of tunnels, 100 miles of covered conduit, 41 miles of lined open conduit, 21 miles of unlined canal, 12 miles of steel siphon pipe, and 882 feet of concrete flume. The first 62 miles of the conduit will have a capacity of 900 cu. ft. per sec. to the Haiwee reservoir, with a storage capacity of 64,000 acre feet, the province of this portion of the aqueduct being to collect ordinary flood waters as well as the normal flow of the streams. From the Haiwee reservoir to the suburbs of the city of Los Angeles, the capacity of the conduit will be 400 cu. ft. per sec., except the power conduit, which will have a capacity of 1,000 second feet. All but the first 21 miles and the steel siphons, will be lined with concrete, calling for a total consumption of 1,200,000 barrels of cement. The construction of the aqueduct is being carried on by the Engineering Department and not by contract.

The aqueduct is located in a sparsely populated region where transportation charges are unusually heavy. A local railroad has been built from Mojave to the intake, and freight charges on this line average 4½ cts. a ton mile, or approximately 1 ct. a barrel per mile. There is a stretch of 60 miles southwesterly from Mojave that diverges from all railroad transportation, at one point the conduit being 35 miles from the nearest railway. The transportation charges on the cement are a more important factor than its first cost.

The city of Los Angeles has constructed a municipal cement mill at Monolith, 15 miles north of Mojave, which is the approximate center of the aqueduct. This mill has now been operating for two years and has a capacity of 1,000 barrels a day. It is of

*Assistant Chief Engineer, Los Angeles Aqueduct, Los Angeles, Cal.

standard construction, making a Portland cement which complies with the standard specifications for cement.* The brand name of the cement is "Monolith." While this cement mill has been successful in producing a product at a reasonable cost near the center of the work, it does not obviate either the high local freight charges north of Mojave, which amount to as much as $1.25 a barrel, or the expensive wagon haul charge to points southwest of Mojave, which, in addition to the local railroad freight, amounts to as much as $1.50 a barrel.

Deposits of volcanic Tufa were discovered along the line of the Los Angeles aqueduct in the Owens valley, 85 miles north of Mojave desert about 20 miles west of the railroad, and at the site of the cement mill at Monolith. Tuff or Tufa is a volcanic, fragmental rock which is made up of minute particles bearing the indications of having been laid down in water and partly consolidated. Sometimes Tufa is a calcareous deposit, but that used for the manufacture of the cement described here is of volcanic origin. It is of a greyish or creamy color, and of a low specific gravity when in the rock form. When pulverized, the powder has a specific gravity of 2.2. It closely resembles and is probably the same as the German Trass and the Italian Pozzuolana, except that these last two are defined as of volcanic origin only, while Tufa is described as being either volcanic or calcareous. The Roman hydraulic mortars consisted of a mixture of pulverized Pozzuolana, slaked lime and sand. The Tufa found along the line of the Los Angeles aqueduct occurs in large deposits and shows lines of stratification, indicating the probability of the ash having fallen on water and been laid down by sedimentation. In texture, Tufa closely resembles the pumice of commerce. Its chemical analysis is

Silica	68.96	per cent.
Alumina	18.05	"
Iron	.40	"
Lime	2.00	"
Magnesia	2.71	"
Sulphur	.35	"
Loss on Ignition	7.63	"
Total	100.10	per cent.

* See Standard No. 1.—Ed.

For a number of years past, the writer has been connected with the construction of hydraulic works in western desert regions where transportation charges often result in high cost of cement. This has led to experiments being made at different points on the possibility of making silica cements by blending ground quartzite rock with standard cement, and later, by grinding and blending volcanic products with standard cement. When investigations looking to the construction of the Los Angeles aqueduct were undertaken, Mr. E. Duryee, who formerly was a cement chemist with the U. S. Reclamation Service, was engaged by the city for the purpose of conducting tests of material along the line of the aqueduct suitable for the manufacture of cement. The blending and grinding of Tufa with cement was suggested by the writer. The development of the process described below was under the immediate direction of Mr. Duryee, assisted by Mr. G. M. Andrus, who at present is the chemist for the city in charge of the tests at all plants. Mr. Roderick MacKay is manager of the cement mill and Tufa plant at Monolith, Mr. J. A. Roesch being in charge of Tufa Plant No. 1 at Fairmont, and Mr. O. J. Riggs in charge of Tufa Plant No. 2 at Haiwee.

The experiment was early made of grinding in the laboratory at the cement mill, the Tufa of the Mojave desert and blending it with hydrated lime. This resulted in a strong, hydraulic mortar, subject to considerable checking when made in standard pats, but a mortar which became hard when set in water. This indicated that there was a chemical union between the ground Tufa and the lime. The experiment of grinding and blending Tufa with standard Portland cement was also made in the laboratory. The laboratory tests showed that briquettes made of the Tufa and cement without sand were not as strong as briquettes made of straight Portland cement, but when the Tufa cement was mixed with standard sand in proportions of 3 to 1, it developed as much strength in 28 days as the standard sand briquettes made with standard cement in the same ratio. Experiments were made with Tufa from the three deposits in which the aqueduct engineers were interested, i.e., the Haiwee deposit in Owens valley, the deposit at Monolith, and the deposit at Fairmont. Practically the same results were obtained from each one. A mill run was then made of the Tufa cement and this was put into the conduit work in the desert under

conditions of extreme summer heat. The work stood up well, making a good showing under actual working conditions, both in the mill and in the field.

After completing these experiments, the first Tufa cement plant was erected at Fairmont at a cost, for all charges, of $22,000. This mill consists of an ordinary rock crusher, with elevators, a ball mill and a tube or pebble mill. This plant is operated by electric motors. The Tufa is quarried, then run through the rock crusher and through the ball mill, where it is passed through a 25 mesh screen. At this point the raw Tufa is blended with the straight Portland cement, and the blended product is then run through the tube mill, where it is ground so that 90 per cent. will pass through a 200 mesh screen. The output of this mill, under ordinary working conditions, will average about 100 barrels of the blended product per eight hour shift. A great deal depends upon the dryness of the Tufa when it is fed into the mill. If the Tufa is but slightly moist, it runs through the fine screen much more slowly than if it is dry. In midsummer when the Tufa is thoroughly dry, the output of the mill may be increased 30 or 40 per cent. with the same operating crew. The Tufa is not artificially dried, heated or burned in any way in the process of manufacture.

As the Tufa is very light and easily crushed, the machinery for the first crushing and grinding of the Tufa probably could have been made less elaborate and expensive than that which was installed. The limiting factor in the product is the fine grinding in the tube mill. It is possible to run the product through the ball mill in about 60 per cent. of the time than it can be put through the tube mill. The mill that has been erected at Haiwee is practically a duplicate of the Fairmont mill, and has the same capacity. The Tufa will be handled in the same manner at the cement mill at Monolith as at the regular Tufa mills.

The following is a cost statement for the month of July, 1910, per barrel of blended product, for operating the Fairmont Tufa plant on a one shift basis:

General expense—Labor, live stock, etc.	$ .04
Power, at 1.85 cts. per k. w. hour	.105
Quarrying	.025
Mill operation	.20
Net milling cost, per barrel	$ .37

The characteristics of Tufa cement, as compared with regular Portland cement in field work are that the Tufa cement requires more water in the mix, hardens more slowly but continues the hardening process for a greater length of time. Tests made in the laboratory of the City Engineer of Los Angeles showed that the Monolith Tufa cement, with one part Tufa cement to three parts of sand, gave a tensile strength in 28 days of 430 lbs., in 90 days, 487 lbs. and in 180 days, 573 lbs. In this instance, the blend consisted of 62½ per cent. of Tufa by volume and 37½ per cent. of Monolith Portland cement.

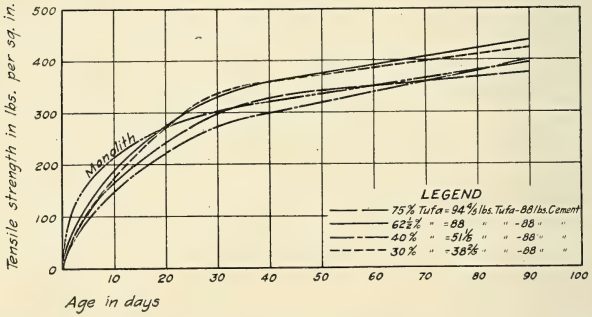

FIG. 1.—DIAGRAM OF TENSILE STRENGTH OF MORTARS OF 1 PART BLENDED MONOLITH CEMENT TO 3 PARTS SAND. PERCENTAGE OF BLEND IS BY VOLUME.

The characteristic of absorbing more moisture and apparently holding it for a longer period of time than is the case with straight cement, makes it a better cement to use in hot, dry weather and not so good a cement to use in cold weather under freezing conditions.

Fig. 1 shows the results of some tests with Tufa cement. The samples used in these tests were actual mill runs of 30 per cent. by volume of Tufa, 40 per cent., 62½ per cent. and 75 per cent. respectively. There is also shown on this diagram, tests made with straight Monolith Portland cement. The blend showing the best

results is 62½ per cent. by volume of Tufa and 37½ per cent. of straight cement. The results, however, are little different from those obtained with 30 per cent. of Tufa and 70 per cent. of straight cement. The Tufa has been ground with a standard Portland cement manufactured in California, and similar results obtained.

The blend that is being made at the Fairmont mill is 50 per cent. by volume of Tufa, and at the Haiwee mill the blend is 60 per cent. by volume of Tufa, the Haiwee Tufa being somewhat lighter than the Fairmont. The tests given in Fig. 2 are the result of averaging a large number of samples. The extreme fineness of

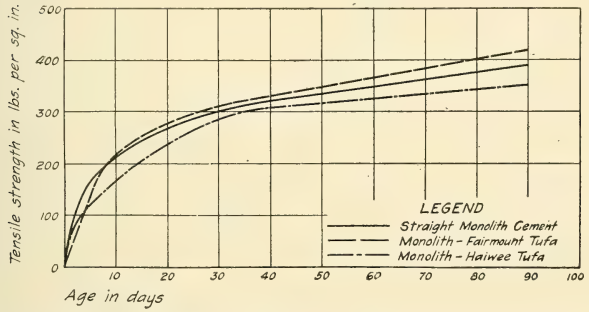

FIG. 2.—DIAGRAM OF TENSILE STRENGTH OF MORTARS OF 1 PART CEMENT TO 3 PARTS SAND. ALL BLENDS 50 PER CENT. BY VOLUME.

grinding and the natural plasticity of the Tufa produce a dense concrete and one that finishes with remarkable smoothness. Tufa cement makes a smoother plaster than straight cement. A disadvantage, in some classes of work, consists in the fact that it attains its strength slowly, so that forms have to be left on the work somewhat longer. The roof forms on the Los Angeles aqueduct, where the clear span is 12 ft., are stripped in 6 days when straight cement is used, and 2 or 3 more days are allowed when Tufa cement is used.

A number of experiments have been made of the effect of adding a small percentage of hydrated lime to the Tufa cement.

The results obtained so far do not justify stating any definite conclusions resulting from such practice.

In addition to the laboratory tests made with the Tufa cement,

TABLE I.—RESULTS OF TESTS IN REINFORCED CONCRETE SLABS MADE WITH GROUND TUFA BLENDED WITH VARIOUS PROPORTIONS OF MONOLITH PORTLAND CEMENT. AGE OF CONCRETE AT TEST: SLABS NOS. 1 AND 2, 31 DAYS; OTHERS, 30 DAYS.

Slab No.	Kind of Cement.	Proportions of Concrete.	Load		Deflection, ins.	Remarks.
			Total lbs.	Lbs. per sq. ft.		
1	Tufa-Portland 30 per cent. Tufa	1:2½:5	5,000	69	.125	
			10,000	138	.313	Crack at 7,500 lbs. total.
			12,500	173	.625	
			15,000	209	.750	
			15,050	.....	.....	Failed. Rods taken out.
2	Tufa-Portland 50 per cent. Tufa	1:2½:5	5,000	69	.063	Crack in center.
			10,000	138	.188	
			12,500	173	.250	Crack at ¼ point to left.
			15,000	209	.375	Crack at 14,000 lbs., at ¼ point to right.
			17,500	242	.438	
			20,000	275	.563	
			25,000	346	.750	
			30,000	415	1.000	
			35,000	484	1.313	
			37,500	519	1.500	
			40,000	558	1.875	
			42,500	588	2.563	No failure. Test discontinued. Permanent set of 1.75 ins. on removal of load.
3	Tufa-Portland 50 per cent. Tufa	1:2½:5	5,000	69	.063	
			10,000	138	.250	Crack at 6,000 lbs.
			15,000	209	.500	
			20,000	275	.688	
			25,000	346	1.000	
			30,000	415	1.313	Poor bearing at right end.
			32,225	434	.....	Broke within 8 ins. of right end.
4	Tufa-Portland 40 per cent. Tufa	1:2½:5	5,000	69	.125	Crack at 1,000 lbs.
			10,000	138	.438	
			15,000	209	.938	
			20,000	275	2.063	
			21,000	288	.....	Slow failure in center.
5	Tufa-Portland 62½ per cent. Tufa	1:2½:5	5,000	69	.250	
			10,000	138	.438	
			15,000	209	.813	
			20,000	275	1.313	
			24,000	334	2.188	
			25,000	346	.....	Slow break. Left end.
6	Monolith Portland	1:2½:5	5,000	69	.188	
			10,000	138	.563	
			15,000	209	1.438	Broke 4 ft. from left end. One end rod pulled out.
			16,500	225	2.375	

a number of field tests were made, the sands and gravels which were used being ordinary material found along the line of the aqueduct. Test slabs were made 6 ft. wide and 12 ft. 5½ ins. long, 8 ins. thick at the center and 6 ins. thick at the ends, with

a clear span of 11 ft. 5⅜ ins. Slab No. 6 was of uniform thickness, 6 ins. The reinforcement consisted of four ⅝ in. square twisted rods spaced 18 ins. center to center, ½ in. from the bottom of the slab at the center, and bent up to the middle at the ends, and 58 ins. and 42 ins. wide mesh reinforcement, 4-in. mesh with No. 12 longitudinal and No. 14 diagonal wires. In slab No. 6 the rods were not bent up at the ends.

The concrete was made of one part, by volume, of Tufa Portland cement (88 lbs. per cu. ft.), and 2½ parts of washed sand containing 42 per cent. voids and weighing 96 lbs. per cu. ft. loose, mixed dry and enough water added to make a plastic mortar. This was spread over 5 parts of crushed granite, the crusher run containing 40 per cent. voids, and weighing 102 lbs. per cu. ft., and the whole turned over three times by hand. The maximum sized rock was 3 ins., and a mixture of 1: 2 of sand and rock contained 28 per cent. voids with a shrinkage of 6 per cent. of the mass. Care was taken to place the concrete well and obtain equal conditions for all slabs.

Before the concrete commenced to dry out, it was covered with a layer of dirt and kept wet for 20 days, after which time the slabs were permitted to dry until they were tested. Two concrete piers 6 ft. long and 6 ins. wide on top were built in a pit 2 ft. deep, so as to give a clear span of 11 ft. 5⅜ ins. The slabs were then rolled over the piers and lowered onto a 1×4, resting on a ½ in. layer of fine sand. By this method, a uniform and flexible bearing was obtained.

In the first tests, the load was applied by placing sand into a wooden frame box, but the sand arched as the slab deflected and the actual load on the slab was indeterminate. In the later tests, water was used. In this way, all arch action of the load was eliminated. The Table I shows the results of the tests, water being used as the load.

Mill runs have been made at the Monolith plant in which Tufa has been substituted for clay in the manufacture of Portland hydraulic cement and a good product obtained. In this case the Tufa and lime were run through the kilns and burned as in ordinary practice of cement manufacture. The clinker resulting was

harder to grind than that obtained by using clay. Microscopic slides have been made of the Tufa briquettes, but their examination does not throw much light upon the way in which the Tufa combines with the cement. It is believed that the beneficial results obtained from using Tufa with cement are both mechanical and chemical. The Tufa will combine with any free lime in the cement, and the fine grinding not only improves the cement, but also permits the fine, silicious Tufa to go one stage beyond the sand in making up the concrete aggregates.

In conclusion it may be stated that, both from laboratory tests and field experience, the Tufa cement has proven as strong and as satisfactory generally as straight cement when used in making concrete. Its advantage on the Los Angeles aqueduct is in the way of economy. The quarry and mill charge for taking one barrel of straight cement and blending it so as to make two barrels of Tufa cement is about 74 cts.; the price of straight cement at the points where the Tufa plants are located is about $2.50.

It must be remembered that these grinding plants are relatively small, operating on a one shift basis, with an output of about 100 barrels per shift, and that if operations were carried on on a larger scale, the cost of grinding and blending this Tufa would be much less, and I see no reason why there should not be marked economy in producing such a blended cement, where Tufa is available.

An elaborate investigation of Trass cements was made under the direction of the Prussian Minister of Public Works and the results appear in a report of the Royal Testing Laboratory under the title *Mitteilungen aus dem Kgl. Materialprüfungsamt.*

These tests show that the Trass mortars consisting of 50 per cent. Trass and 50 per cent. Portland cement, when mixed with sand and tested for tensile strength in standard briquettes, continue to harden after one year's time, the greatest strength being developed in the five year or oldest samples. They also show that the Trass cement is slower in hardening, but ultimately attains a strength fully equal or superior to the straight cement and sand briquettes, when the volumes of Trass cement and straight cement are equal. The higher the proportion of sand, the better is the relative showing made by the Trass cements.

The following data* show the results of the Prussian tests:

AVERAGE STRENGTH OF MORTAR AND CONCRETE SPECIMENS
Stored in Fresh Water. Stored in Sand 9 Days.

Proportions	Age	Tensile Strength in lbs. per sq. in.
1 cement, 2 sand	28 days	572
	1 year	722
	5 years	722
1 cement, 4 sand	28 days	348
	1 year	426
	5 years	362
1 cement, 1 Trass, 4 sand	28 days	504
	1 year	616
	5 years	722
1½ cement, 1 Trass, 5 sand	28 days	573
	1 year	640
	5 years	724
1 cement, 1 Trass, 8 sand	28 days	242
	1 year	353
	5 years	399
1½ cement, 1 Trass, 10 sand.	28 days	240
	1 year	320
	5 years	329

Unfortunately, nothing is given in this review regarding the fineness of the grinding of the Prussian samples. In the work on the Los Angeles aqueduct, this has been determined to be an important factor. In general the conclusions of the Prussian investigators and of the Los Angeles aqueduct engineers closely resemble each other, though they were entirely independent, as the report on the Prussian tests only came to our knowledge very recently.

As volcanic formations occur frequently in the western portions of this country, the use of Tufa in the manufacture of cement may have general commercial importance on account of its economy.

*_Engineering Record_, August 27, 1910.

DISCUSSION.

Mr. Spackman. MR. H. S. SPACKMAN.—I have made a number of tests on a mechanical mixture of Tufa and lime which gave remarkably good results. One day in air and the balance of the time in water, a mixture of cement and sand, 1:3, when tested in tension after 7 days had a strength of 58 lbs. per sq. in.; at 28 days, 205 lbs.; at 3 months, 266 lbs.; and at 6 months, 285 lbs.

RESOLUTIONS RELATIVE TO THE TRANSPORTATION AND EXAMINATION OF TRASS.

Society of Hungarian Engineers and Architects, Budapest, November, 1909.

Section 1.

Materials Producing Hydraulic Properties.

1. All natural and artificial materials which will harden, both in air and water, when mixed with lime are designated Materials Producing Hydraulic Properties. Such materials are the whole series of Tufa of recent volcanic eruptions; among which are the Puzzuolana, Santorin earth, slag, as well as various other mixtures of clay and silica.

Trass.

2. Volcanic Tufa containing silicic acid in sufficient proportion is designated Trass.

3. Trass, the same as most materials producing hydraulic properties, is used in the form of finely ground powder; it comes on the market either in rock or powdered form.

Use of Trass.

4. The mortar or concrete obtained by mixing lime hydrate with Trass is used to great advantage chiefly in hydraulic construction.

Section 2.

Transportation.

Packing.

1. The powdered Trass must be packed in bags or barrels; the price should be fixed per 100 kg. (220 lbs.).

Weight.

2. The weight of each bag of powdered Trass should be 50 kg. (110 lbs.) net; a variation of 2 per cent. shall not be objectionable. The weight of the bag must not exceed 1.5 per cent. of the gross weight.

3. Each bag or barrel must be plainly marked with the name of manufacturer, in addition it must bear the words "Powdered Trass" and be labeled with the net weight. The bags and barrels must be sealed by means of lead seals, on one side of which the

name of manufacturer must appear, while on the other the words "Powdered Trass."

Responsibility. 4. The manufacturers shall be responsible for the good quality as well as for the quantity of all shipments of Trass.

SECTION 3.

Examination of Trass. 1. The quality of Trass shall be determined by the following properties: Fineness of grinding, loss on ignition, quantity of soluble silicic acid contained, length of time for setting, constancy of volume and strength of binding.

SECTION 4.

Fineness. 1. The Powdered Trass should be such that when passed through a sieve containing 900 meshes per sq. cm. (about 76 meshes per lin. in.), thickness of wires being 0.10 mm. (0.00394 in.), the residue shall not be more than 10 per cent. of total weight; on a sieve containing 4900 meshes per sq. cm. (178 meshes per lin. in.), thickness of wire being 0.05 mm. (0.00197 in.), the residue shall not be more than 30 per cent. of total weight; that is to say, the total residue of the two siftings should not be more than 40 per cent..

The test must be made with 100 gr. (3.52 oz.) of material and the average of three tests shall be taken as the result.

2. The sample of Rock Trass for tests must represent the average quality of the delivered rock; the sample shall be ground to the required fineness and the powder thus obtained examined.

SECTION 5.

Loss on Ignition. 1. About 20 gr. (0.7 oz.) of the Trass powder should be heated and kept at a temperature of 98 deg. C. (208 deg. F.) until there is no appreciable loss in weight due to heating.

A quantity of 10 gr. (0.35 oz.) of the Trass powder should be ignited in a porcelain or platinum vessel by means of a blowpipe or furnace in such manner that in 10 minutes the powder should be at red heat, and such heat maintained for at least 40 minutes.

2. The ignited Trass powder should be cooled in the exsiccator, afterwards reweighed, and the loss in weight (due to water hydrate) thus found should be at least 7 per cent.

SECTION 6.

1. Trass powder when dried at a temperature of 98 deg. C. (208 deg. F.) and treated with a solution of calcium hydroxide should contain at least 20 per cent. of soluble silicate. **Silicic Acid.**

SECTION 7.

1. The binding quality of Trass powder must be determined on a mixture consisting of 2 parts of Trass powder and 1 part powdered lime, by weight. **Time of Setting.**

2. The powdered lime should be such as may be obtained by a proper burning process from a pure quality of limestone free of clay. It should be prepared so that the powder shall contain about 24 per cent. of water by weight; (*i. e.*, 18 parts to 74). Its fineness should at least agree with that of the Trass powder. *Powdered Lime.*

3. The necessary powdered lime for this purpose is obtained by means of drying out ordinary quick lime of a pasty consistency, free from carbonic acid, taken from the pits and grinding same, thus reducing the possibility for unburned particles remaining in the powder. The powdered lime intended for these tests should be prepared under the supervision of the State Experiment Station.

4. The setting shall be determined by means of the Vicat needle; the above mixture of the required consistency should be used. *Vicat Needle.*

5. The quantity of water required (approx. 30 to 40 per cent.) to obtain normal consistency shall be determined by means of "determination of consistency" as directed in the Resolutions Relative to Transportation of Cements of the Society of Hungarian Engineers and Architects (C, 2).* *Normal Consistency.*

6. This mixture of Trass having a consistency of paste usually begins to set 30 minutes after the addition of water; complete set may not take place for hours and days, hence a preparation of Trass of this consistency is as a rule slow setting.

SECTION 8.

1. The constancy of volume shall be determined on test pats made to proper consistency as per Section 7, 1. **Constancy of Volume.**

*See p. 485.—ED.

The method of procedure in other respects is the same as that prescribed for cements in Resolutions (C, 3).*

2. If any changes in the test pats should take place with respect to volume (if cracks, bulgings appear) whether kept in air or under water, such deformations shall indicate that the line used was not suitable for the purpose.

SECTION 9.

Strength. 1. The strength of binding is determined on a mortar consisting of 2 parts Trass powder and 1 part powdered lime and 3 parts of standard sand, and from 0.9 to 1.0 part water, each taken by weight.

2. The mortar required for each of the test pieces should be so divided that when placed in the forms and even after the tamping, the mortar may extend over the top of the form by one to two (1 to 2) (about 1/16 in.) mm.; this 1 to 2 mm. should be removed when smoothing off.

Test Pieces. 3. The test pieces for tension must be forced into the form with 120 blows of the 2 kg. (4.4 lbs.) ram, and for compression, the material should be subjected 150 blows of the 3 kg. (6.6 lbs.) ram.

Water. 4. The quantity of water should be such that after the ramming process has been concluded, a little thin slime should be perceptible on the surface.

As to the quality and preparation of sand the remarks in Resolutions regarding Portland Cement shall apply (C, 5 and 6).†

5. The test pieces after having been rammed, must be left in the mold until completely set, and on removal should be kept in a moist atmosphere for one day, after which placed in water of 15 to 18 deg. C. (59-65 deg. F.), changing the water each week; the pieces remain therein until the time of testing.

6. Of a total of six tests the average of the four best ones shall be taken as the result. The average compressive strength at the end of 28 days should be at least 60 kg. per sq. cm. (853 lbs. per sq. in.).

*See p. 487.—ED.

† See pp. 489-494.—ED.

SECTION 10.

1. During the examination of cements, the weight per liter of the Trass powder should be determined as follows: loosely placed, sieved and also well shaken. Its specific gravity should also be determined. **Weight per Liter.** **Specific Gravity.**

Accepted by the Society of Hungarian Engineers and Architects at a regular meeting (512), held August 4, 1909, Budapest.

P. BALLA,
First Secretary.

M. KAJLINGER,
President.

THE RELATION OF THE LIME CONTENT OF CEMENT TO THE DURABILITY OF CONCRETE.

By H. S. Spackman*.

The purpose of this paper is to bring to your attention the discussion on the effect of the high lime content of cement on the durability of concrete construction under special conditions, which has occupied more or less the attention of technical societies and the columns of the technical press, especially in Europe, during the past few years.

This paper is not to be interpreted as questioning the advisability of the use of concrete, or distorted into a prophecy of the failure in the near future of the numerous concrete constructions in existence, for concrete, when properly prepared, is one of the best and most durable of structural materials.

It is desired, however, to emphasize the fact that its durability is not due to the Portland cement content, but to the care in the selection of the aggregate, the proportioning of same, and the workmanship in mixing and placing the concrete. The making of concrete is a manufacturing process, in which the cement is only one of the several raw materials required, and forms but a small proportion of the finished product. Such being the case, the durability and strength of the finished material depends on the user, and not on the cement manufacturer; and this will continue to be the case until there has been developed a hydraulic cementing material, which will alone be capable of resisting the destructive action of water containing sulphates, or other injurious chemicals.

This discussion may be epitomized in the following statements:

1. All Portland cement is, by reason of its high lime content, subject to destructive attack, under conditions favorable to such action, not only from sea-water, but also from acidulous and other saline waters.

2. The resistance of concrete to the disintegrating effect of such waters depends largely upon the impermeability of the mass.

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3. That it is essential to the durability of concrete construction under unfavorable conditions that there be developed a hydraulic cement having strength under mechanical test approaching that of Portland and free from the properties which render it liable to destruction by chemical action under such conditions.

Owing to the large use in Europe of cement in maritime construction, where the evil effect of the high lime content is accentuated by exposure to sea-water, more thought has been given in Europe to this particular phase of the question than here, or than to the study of the destructive effect of ground waters, which is equally noticeable, and even more important. Enough work, however, has been done along these lines to show the danger of the use of high lime cements when the construction is to be subjected to waters containing sulphate of lime, which Bied, in his interesting paper *Experiments on the Decomposition of Mortars by Sulphate Waters,* read at the Fifth Congress of the International Association for Testing Materials at Copenhagen in 1909, shows to be even more destructive in their effect than sea-water.

The importance to all constructors of the fact that waters containing solutions of sulphate of lime exert a destructive influence on Portland cement concrete, becomes manifest when it is realized that the ground waters of our large cities generally contain considerable amounts of sulphuric acid, which, on contact with the cement, may dissolve the lime and form destructive solutions of sulphate of lime.

Bied states in the article referred to:

> The decomposition of mortars, which are exposed to the action of the sea or of waters charged with sulphates, has for a long time already occupied the attention of engineers, and experiments have been multiplied in all quarters with the object of discovering some hydraulic binding material which would be free from liability to destruction.
>
> Up to recent years attention has chiefly been concentrated on the injurious action of the sea-water, and although the destructive effect of sulphate of lime was not unknown, it was not as a rule taken into consideration.
>
> Having had to inquire into decomposition of mortars by sulphate of lime, observed in 1900, on the line leading from Linarez to Algeria in Spain, and in 1902 on the line St. Girons-Foix, and at the same time in Algeria, we have been induced to submit this problem to a prolonged experimental investigation.

In our researches we have compared materials poor in alumina and aluminiferous mortars, highly calcareous mortars and some of high index.

The action of the diverse saline solutions which were employed is not altogether the same. It may in general be assumed that the artificial sea-water is 10 or 15 times less active than a 12 per mille solution of magnesium sulphate. Yet certain artificial cements have been decomposed more rapidly in sea-water than in the solution of anhydrous magnesium sulphate.

The saturated solution of sulphate of calcium is at least as destructive as the 12 per mille solution of anhydrous magnesium sulphate, but its effect appears to be different, sometimes more energetic, sometimes less energetic than that of the magnesium sulphate solution of 12 per mille.

Water saturated with sulphate of lime appears to be the most active decomposing agent. Constructors must constantly be on the alert in this respect, in fact, waters saturated with calcium sulphate are sometimes met with far from any recognized occurrence of gypsum.

On the other hand, waters which at certain moments do not contain more than 0.1 or 0.2 gram of calcium sulphate, may become saturated during periods of drought.

In our laboratory and under the experimental conditions above indicated, we have not met with any slow or quick-setting cement, whatever its mode of manufacture and its chemical composition, which has resisted the action of this solution for six years.

Similar destructive effects of the ground water have been noted in the West where the water is charged with a solution of alkaline salts.

Whether or not the same rule, the lower the lime content, the more durable the cement, will hold good for cements above ground, exposed only to the influences of the atmosphere and the moisture and gases contained therein, I am unable to say, as I cannot recall any careful and systematic investigation and study of the comparative durability and strength over long time periods, where low lime and high lime cement concretes were tested under like conditions, that is, in air, in a dry state under the conditions similar to those occurring in the interior of reinforced concrete construction, and also exposed to alternate wetting and drying, as would be the case of the concrete on the exterior of reinforced concrete construction. Such an investigation extending over a period of several years where a number of

full-sized beams were tested at yearly intervals, the test pieces being made from the same aggregate with various cements, would be of great value.

It is known, however, that the briquettes of Portland cement stored in air show greater variation and more often retrogression of strength and disintegration than companion comparison sets of briquettes stored in water. It is also known that Portland cement clinker will decrepitate with time when exposed to ordinary atmospheric influences, and even in sealed air-tight bottles, and that the higher the percentage of lime in the clinker the more marked is this disintegration. It is also known that only a portion of Portland cement ever hydrates or takes part in the hardening. It is, therefore, not unreasonable to suppose that this unacted upon clinker in the cement will disintegrate with time. Whether the disintegration of briquettes stored in air, or the loss in strength, be due in part to the decrepitation of the coarser particles of cement as above indicated or not, is at present a matter of speculation, but it would seem, in view of the well known action of clinker that it would be desirable to have as little inert material in the cement as possible.

That Portland cement is not of itself sufficient under ordinary conditions to insure impermeability in concrete is evidenced by the rapidly growing use of waterproofing compounds, either mixed with the cement or applied as a surface coating, and it would seem that there should be an endeavor, either by improvement in the quality of the cement used or in the workmanship and proportioning of the concrete, to make an impervious material, as it is unquestioned by all investigators that the resistance of concrete construction to the attack of acidulous or saline waters, or other destructive influences, is entirely a function of its permeability.

Are constructors justified in the face of these facts in confining themselves almost entirely to the use of Portland cement to the exclusion of the lower lime hydraulic cements, as it is evident from the report of the Geological Survey covering the cement industry for 1909, which shows that there was produced and used in the United States during 1909, 62,508,461 barrels of Portland cement against the production of 1,687,925 barrels of hydraulic cements of all other classes.

The impression must not be gained from this paper that the destructive action is quite general and that Portland cement is, therefore, unsuited for use under such conditions. As evidence that concrete can be successfully used under adverse conditions, we have properly built structures that have been subjected to such conditions for many years.

It is, however, desired to emphasize that great care must be taken in the use of Portland cement. It is evident that the ability of Portland cement to resist such action can be very much improved, and it was for the purpose of directing attention to this fact, and suggesting that the subject be further investigated to determine whether perhaps some of the other classes of cement might not be more suitable for such use that this paper is written.

DISCUSSION.

Mr. Lewis. Mr. Myron H. Lewis.—I would like to inquire whether Mr. Spackman has experimented with a cement very similar in composition to that referred to and is said to contain 42 instead of the usual 65 per cent. of lime.

Mr. Spackman. Mr. H. S. Spackman.—I have made no investigations in connection with sea water, but am interested in the subject and presented a summary of recent discussion on the subject.

Mr. Chapman. Mr. Cloyd M. Chapman.—The paper intimates that waterproof concrete is not made with Portland cement and I would like to ask whether the writer believes that it is impossible to make a waterproof concrete with Portland cement?

Mr. Spackman. Mr. Spackman.—The object of the paper was to show that concrete cannot be made from poor or improperly sized aggregates, relying on the cement to give it strength and durability, as the cement itself would be acted upon by injurious elements. The durability of a concrete is dependent on making it waterproof through the proper selection and proportioning of the aggregates, workmanship and care in placing, and not on the cement.

I absolutely believe it possible to make concrete waterproof. Our difficulty in the laboratory in making permeability tests with a 1:3 mixture is to make one that is not waterproof.

Mr. Wig. Mr. Rudolph J. Wig.—The statement was made that the permanency of concrete in sea water was due to the density and and impermeability of the concrete to the water. In our tests the water penetrated entirely through the concrete in 8×16 in. cylinders in a very short time between the tides where the water has a severe pounding action, while it did not penetrate more than 2 or 3 in. in similar concrete totally immersed for a year where we had no apparent disintegration.

Mr. Spackman. Mr. Spackman.—I think, considering all sea water tests, there has been a great lack of uniformity. Water going through a block or not going through, does not necessarily mean that the mortar is porous. Water will go through a sieve, but the wires

Mr. Spackman. of the sieve may not necessarily be porous. It seems to me that in order for marked destructive action from the sea water to take place it must get in contact with each particle of the cement. If the cement prevents the water getting access to the interior and the destructive action is limited only to the surface it must be very slow, especially where it is not assisted by mechanical abrasion. Now the question of the amount of water passing through concrete, or not, does not necessarily show the permeability of the mortar itself which binds it together. You can imagine a concrete made of very large spheres which were only in contact at certain points and through which water would pass readily. Possibly if you took a section it would be like a lattice work, yet the cementing portion might be absolutely impermeable and it would not be affected by sea water.

Mr. Wason. MR. L. C. WASON.—I built 24 specimens which were immersed in sea water at the Charlestown Navy Yard, Boston. Some of those were designed porous, so as to be acted upon by sea water promptly, and others to be dense. I took the trouble last Saturday, just before coming over here, to go and look at them carefully at low tide. The three which were made of average normal Portland Cement but made extremely dry so as to be admittedly porous, after not quite two years, one summer and one and a half winters, had shown but little sign of disintegration; but every other specimen whether made of cement with high alumina or low alumina or without alumina or without iron, was apparently perfect.

THE EFFECT OF ELECTROLYSIS ON METAL EMBEDDED IN CONCRETE.

By Cloyd M. Chapman.*

Portland cement mortar and concrete form an excellent protecting cover for iron and steel under ordinary conditions, but when concrete has steel embedded in it under such conditions that there are electric currents passing through the steel and from the steel to the concrete, there is set up a corrosive action due to electrolysis that may rapidly deteriorate both the metal and the concrete.

The conditions which bring about this destructive action are neither uncommon nor complicated. If electric currents pass into dense concrete through steel which is tightly embedded in it, an action takes place which will in time disrupt the surrounding concrete. If the concrete is not dense, but contains an appreciable percentage of voids and is therefore somewhat compressible, as in the case of a cinder concrete, then the result of the swelling action may be that of closing up some of the voids and so no actual rupture of the mass may occur. If the steel but rests upon the surface of the concrete, and is not embedded in it, the action may take place on the plane of contact without disrupting the concrete. The current passing may be extremely small, with the result that the action is very slow, but the effect is the same in the end.

It is not uncommon practice to embed the bases of steel building columns in concrete, or the bases of elevated electric railway structures, and bridge and trestle columns. In any of these cases there may be small stray electric currents passing from the metal to the surrounding concrete if electric currents are much used in the building or if electric railways pass over the bridges or trestles.

If the conditions are fulfilled, that is, if a portion of the steel is solidly embedded in dense concrete and if even small electric currents pass through the steel and leave it and enter the concrete

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where it is thus firmly embedded, it is only a matter of time until the concrete will crack and break away from the steel.

These facts have been well established. They have been previously studied, experiments demonstrating them carried out, and results reported in several technical publications during the past four years. Some of these investigators reported their work in the following publications:

Maximilian Toch. Journal, *American Electro-Chemical Society*, June, 1906.

A. A. Knudson. Journal, *American Institute Electrical Engineers*, 1907.

N. J. Nicholas. *Engineering News*, Dec. 24, 1908, and Dec. 1, 1910.

A. S. Sangsdorf. Journal, *Association of Engineering Societies*, Feb., 1909.

G. B. Shaffer. *Engineering Record*, July 30, 1910.

Some further data bearing upon this destructive action was obtained by a series of experiments conducted by Westinghouse, Church, Kerr and Company, and which are described as follows:

A series of steel plates about 1/16 in. thick, 2 ins. wide and 6 ins. long were embedded in concrete blocks as shown on Fig. 1. These plates had been previously painted with two full flowing coats of paints of various kinds. In each concrete block were placed 11 plates, 10 of which were painted and 1 uncoated. The purpose of painting the plates was to seek a proper protecting medium for steel exposed to these conditions, but the results so far as they have to do with protective coatings will be reported at another time.

The concrete blocks were about $6 \times 6 \times 14$ ins. They were made of dense concrete, using a mixture of samples of good brands of American Portland cement, and, for the coarse aggregates, crushed limestone, crushed granite, gravel and cinder. Two sets of blocks were made up. They were made of the same mixtures and in them were embedded plates coated with the same points. The one difference in the treatment of the two sets of blocks was that a small electric current was passed through the plates in the one case and not in the other. In both cases the blocks were kept standing in a shallow metal pan of water to keep the concrete damp.

As shown in Fig. 1, all the plates were connected in parallel

to the source of the current and the direction of flow of the current was from the plates through the concrete to the metal pan. The voltage was maintained at about 10 by means of the lamp bank shown. The total current flowing was less than two-tenths of an ampere.

The first effect apparent on the blocks subjected to electrolysis was the appearance around the plates where they entered the concrete, of small brown globules or bubbles, apparently of iron oxide. These may have been caused by the escape of small quantities of gas liberated by electrolytic action on the surface of the plate down in the concrete, which has traveled up along the plate, and, carry-

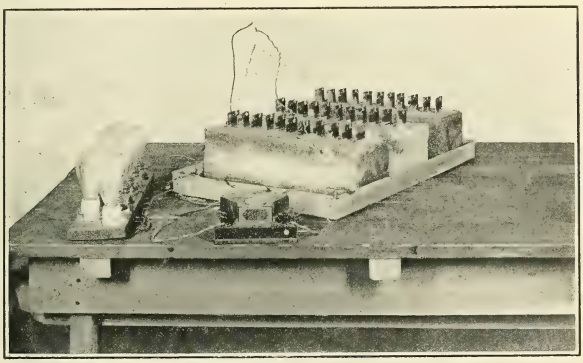

FIG. 1.—ARRANGEMENT OF APPARATUS FOR TEST.

ing iron oxide with it, has left a deposit of rust where the gas escaped on the surface. In Fig. 1 the arrow *A* points to the spot where these globules appeared. This excrescence made its appearance after one to three weeks and gradually increased. Its amount seems to be proportionate to the amount of corrosion taking place on the plate about which it forms. Those plates which corroded very little showed no such excrescence as those which were badly affected.

The next noticeable effect was a very fine hair crack on the top of the concrete block, extending from one or both edges of the plate outward. Gradually this hair crack grew until it reached the sides of the block and finally the block split through completely.

This bursting of the block has occurred only in cases where the corrosion has been very bad and has taken place at periods ranging from a week to three months or more after current was turned on. In Fig. 1 at the arrow *B* appears one of these cracks and Fig. 2 shows it in detail. The tests were continued until the blocks began to crack and split. They were then broken open and the plates removed. These results took place only in the set of tests which were subjected to the electric current. The other set showed no

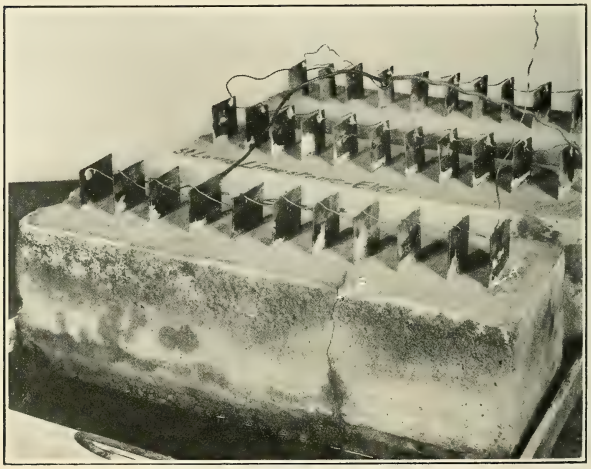

FIG. 2.—BLOCK SHOWING NATURE OF CRACK AFTER ONE WEEK.

change in appearance whatever on the outside of the concrete block. In these the plates were all free from rust or corrosion except at the point where they entered the concrete and where the metal was subjected both to the moisture of the concrete and to the air. Here and there appeared some slight rusting. The concrete was all sound and hard.

In the case of the blocks subjected to the electric current the corrosion of the metal was very marked and the plates were deeply pitted. The denser blocks were broken in many places while the porous cinder concrete block was still sound and whole, although

the corrosion of the metal was as marked as in the other cases. Fig. 3 shows one of these blocks after two months' exposure. Not all of the cracks were large enough to show in the photograph but they had developed at seven of the eleven plates. So far these experiments and results agree with those described in the articles mentioned above. But here the close resemblance ceases.

When the cracked blocks were broken apart the concrete was found to be hard and sound, except immediately adjacent to the metal plates where the concrete seemed slightly softer to a depth of perhaps 1/16 in. There was no appreciable spreading of iron

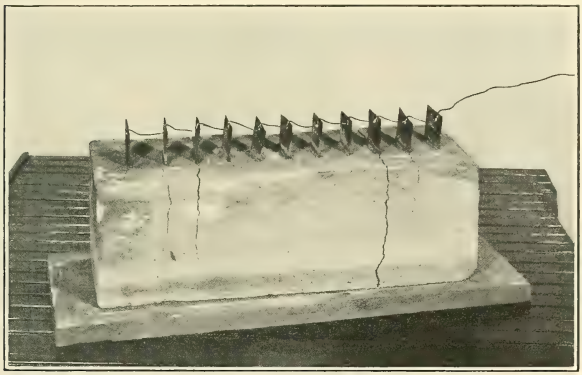

FIG. 3.—CRACKS AFTER EXPOSURE OF TWO MONTHS.

rust along the fracture, which showed clean fresh concrete except for some discoloration extending very slight distances from the metal, $\frac{1}{8}$ in. or less.

As to the cause of this rupture of the concrete five different theories have been offered as follows:

1. Expansion of the concrete due to change in its chemical nature.
2. Hydrostatic pressure.
3. Pressure of liberated gases.
4. Internal heating and consequent expansion.
5. Expansion of metal due to corrosion.

It will be noted that only one of these theories, the last one, depends upon the presence of a metal. Each of the others would hold equally well if some non-metallic conductor were embedded in the concrete and a current passed.

In order to get further evidence as to which of these five explanations is the nearer correct, the following experiment was carried out.

Two 6-in. cubes of dense concrete were made up, in one of which was embedded a ½-in. round iron rod and in the other a ½-in. round carbon rod. Carbon was chosen because under the tempera-

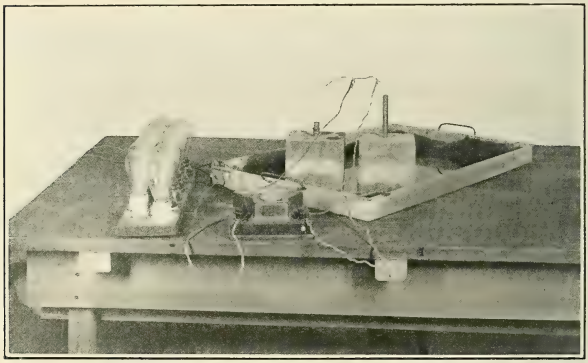

FIG. 4.—ARRANGEMENT OF APPARATUS FOR COMPARING EFFECT ON STEEL AND CARBON RODS.

ture and conditions it would not oxidize or swell appreciably, yet it would transmit current to the heart of the concrete block just as the iron rod would and offer little resistance. Electrolysis would take place on the surface of the carbon as it would on the surface of the iron, gas would be formed in the one case as in the other; in fact, the only important difference in the action between the carbon and the iron rods, for the purpose of this investigation, is the fact that the iron oxidizes or rusts under these conditions while the carbon remains passive and unaffected. These two blocks were then connected up in parallel and a current passed through them, as shown in Fig. 4. There was no rise in tempera-

ture which the hand could detect. A difference of potential of about 10 volts direct current was maintained on the two blocks. In just one week the block containing the iron rod burst and after a period of over two months the block containing the carbon rod was apparently as sound as ever. The voltage was then raised to 30 and two days after to 50 with no apparent effect. Fig. 5 shows the condition of the block containing the iron rod after about two weeks' exposure to the action of the electric current at 10 volts.

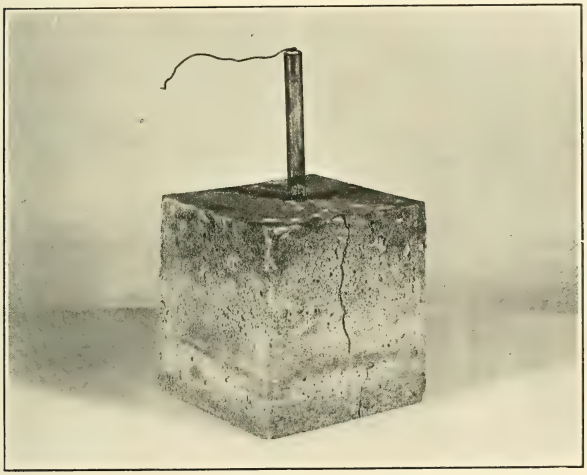

FIG. 5.—CRACK CAUSED BY STEEL ROD ON EXPOSURE OF TWO WEEKS.

In the light of these results the theories enumerated above may be examined to ascertain which of them seems to most fully explain the phenomena described.

1.—The first mentioned theory, namely, that the cracking is caused by the expansion of the concrete due to change in its chemical nature is commented on by its advocate as follows:

> Experiments . . . seemed to show that . . . the strong oxidizing agents, chlorine and oxygen, permeated the pores of the concrete and in the presence of water, changed some of the cement compounds to higher states of oxidation. Such a change should make more space necessary and so act as a bursting force.

This may be answered with the statement that with the carbon electrode the same gases would be liberated and the action of these gases upon the concrete would be the same, therefore the concrete should have burst as surely with the carbon rod as with the iron rod. The fact that this was not the result throws doubt upon this theory.

2.—The second theory is that a hydrostatic pressure is produced at the anode which ruptures the block. An advocate of this theory says:

> According to the tests of Quincke . . . it appears that an actual bodily transfer of electrolyte takes place under the action of electric currents. . . . This transference caused a rise in hydrostatic pressure at the anode or cathode by heaping up the electrolyte, as it were.

If this were true when iron is used, there appears no reason why it should not be equally true when a carbon electrode is used, and a theory which does not explain this discrepancy is defective.

3.—Of the third theory, which accounts for the cracking by the presence of a gas pressure at the anode, an exponent says:

> The cracks in the concrete are best accounted for by the following theory of gaseous pressure: The steel pipe . . . liberated two gases at its surface, viz., oxygen and chlorine. If these gases were hindered in their escape by the surrounding concrete they would accumulate and gradually increase in pressure. As the pressure in a particular void increased, the surrounding cement crystals would be subjected to a tensile stress . . . and might reasonably crack.

To this the same answer applies, namely, that the gases are liberated by the carbon rod as by the iron rod.

4.—The fourth theory mentioned is to the effect that the current heats up the concrete internally and the consequent expansion bursts the block. Its sponsor says:

> This leads to the belief that the indirect thermal effect of the current is a more important factor than the formation of gases under sufficient pressure to rupture the concrete. . . . The temperature may be supposed to have risen considerably along the path, giving rise to local expansion, cracking and general disintegration.

As to the former theories, the reply again is that this heating by current is in no way peculiar to an iron electrode and therefore the block containing the carbon rod should burst as quickly as the

one having the iron rod. Even the increase in voltage from 10 to 50, with the consequent increase in current passing, and in the heat generated, apparently had no effect on the block, though the iron burst its block in one week at 10 volts.

5.—The fifth theory, which is the one most commonly advanced, seems to explain more completely the phenomena noted. We all know that iron rust occupies more volume than the original metal from which it was formed, and as the rust which formed on iron subjected to the conditions here described must find space for itself, a great internal pressure is set up as the amount of rust increases until the enclosing and confining concrete gives way.

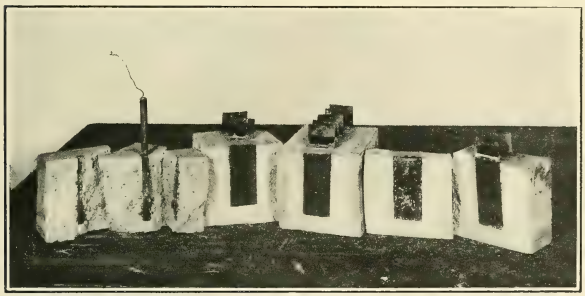

FIG. 6.—TYPICAL FACES OF CONCRETE RUPTURED THROUGH CORROSION OF STEEL.

This great internal pressure also explains the partial breaking down, or crushing or softening of the layer of concrete immediately adjacent to the metal. Fig. 6 shows typical faces of concrete which have burst due to electrolytic corrosion of iron embedded in it. Some of these plates were painted and some were not. It will be noted that part of the paint adheres to the concrete. In the case of the faces in this picture where the plate was unpainted and the corrosion was very rapid, there is no spreading of the iron oxide away from the plate although some rust adhered to the concrete when the plate was removed.

It is not only when a current is passing that steel corrodes when embedded in concrete. Reinforcing bars are sometimes placed so

close to a surface exposed to the weather that air penetrates the concrete to the steel in dry weather and water reaches it in wet weather. Under these conditions the concrete does not possess sufficient protective power to overcome the corroding tendencies of steel which is alternately wet and dry. This becomes especially true after repeated rains have washed out a portion or all of the free lime in the outside layer of the concrete which so effectively

FIG. 7.—EFFECT OF CORROSION ON REINFORCEMENT NOT SUFFICIENTLY EMBEDDED IN CONCRETE.

protects iron and steel from corrosion. In outside walls the reinforcing steel should be kept far enough from the surface to be well protected. As an illustration of the failure that may follow the placing of steel too near the surface, there is shown in Fig. 7 a photograph of an exterior wall in which the steel in a few places was less than ½ in. under the surface. The result was the corroding of the metal first at the points where the rods were nearest the surface and then the gradual extending of the corrosion along the rods, with the consequent bursting out of the concrete in which

it was embedded. In time this action may lay bare the entire length of the rod. This building was erected about nine years ago and the rusting of the bars and cracking out of the concrete became apparent after some two or three years.

Similar effects have followed the use of ungalvanized metal lath over which insufficient coating of mortar had been placed. In some cases large areas have scaled from outside stucco walls because the rusting of the iron lath and the consequent swelling action separated the stucco on the outside of the lath from that on the inside. Having thus lost its bond the outside stucco falls off very easily. The painting of metal lath to protect it against such action we believe to be of little avail. The remedy lies in the use of a substantial, well galvanized lath and placing on it a good and sufficiently heavy coating of dense plaster.

DISCUSSION.

Mr. Bates.

Mr. P. H. Bates.—Mr. Chapman made some remarks about the various theories advanced to explain the distintegration of concrete due to electrolysis, but apparently did not seem to think that the phenomena at the basis of these theories were observable or would explain the destruction.

However, the theory of hydrostatic pressure and the concentration of a constituent of the cement around one of the poles can easily be demonstrated by passing the current through a long narrow piece of concrete (about 12 ins. long by 2 ins. in cross section), on the two ends of which two pieces of glass tubing of a narrow bore have been inserted. You will observe on partly filling the tubes with water that the level falls in the one and rises in the other. I have had this column rise in one case to the height of 18 ins. when it overflowed the tube.

Also, if you will analyze the concrete around the poles which you have used in electrolysis, you will find that the SO_3 has been carried contrary to the current and has been concentrated around that particular pole. This is quite noticeable, though it is possibly not sufficient to account for disintegration.

In the case of cinder concrete, you can have a current set up in the concrete, as most cinder contains unburned coal which with the steel will form a couple, producing a slight current. By the use of the electrodes, other than steel, the speaker almost conclusively proves that the disintegration is due to the solution of the steel electrodes and the passage of the iron from it into the concrete where it is thrown out in a voluminous mass of hydrated iron oxide which no doubt causes the swelling and destruction. However, the above-mentioned phenomena, which he did not seem to think took place, do actually occur.

Mr. Chapman.

Mr. Cloyd M. Chapman.—I did not mean to give that impression, as I believe these actions do take place, but not to a sufficient extent to cause rupture. I am leaving the carbon rod

in the block with the current passing and shall increase the voltage up to 110, and it may break. I do not think any one of the theories, except the one pointed out in the paper, fully explains the common rupture of concrete. Mr. Chapman.

The President.—You have made no experiments on concrete in which sufficient age had been obtained to eliminate a large percentage of the moisture, *i. e.*, no tests were made of old concrete where it could be reasonably supposed that the excess of water had evaporated. The point I wish to make is that while this electrical action may occur in the early days of the concrete structure, it is probable that with the thorough drying out of the concrete in the building the electrolytic action would disappear. The President.

Mr. Chapman.—The reason I did not use dried test pieces was because on reading up the subject I found it had been well demonstrated that a dry concrete is a very good insulator and that it requires the presence of water to bring about electrolytic action. When concrete is dry it acts as a conductor and not as an electrolyte. The trouble occurs usually in bases of columns in buildings which are usually sufficiently surrounded by damp earth to be kept damp forever. The absorption of moisture from damp ground would be sufficient to cause the concrete to act as an electrolyte rather than as a conductor. The bases of railway bridges, trestles and structures out of doors are, of course, always damp, but if the concrete were to become quite dry after ageing I doubt very much if the action would continue. Mr. Chapman.

The President.—It would seem that the conditions set forth in the paper would be dependent largely on exposure of metal, so that there would be an opportunity for this action to take place; in other words, if the imbedded metal was protected by a dense mass of concrete of sufficient thickness, that action would hardly be as intense as it was in the tests reported. The President.

Mr. Chapman.—If the current is prevented from reaching the steel the action would be greatly reduced. The current must get into the metal first, and it is only at the anode, or where the current leaves the metal, that the action takes place. Where it enters there is no appreciable action. Mr. Chapman.

The President.—In other words, when the metal is properly insulated with concrete the dangers are not great. The President.

Mr. Anderson. MR. ROBERT ANDERSON.—An interesting experience with electrolysis occurred in the case of a reinforced concrete slab to the bottom of which a zinc sheet was cemented with a plaster of some kind. The zinc was all consumed and the steel was oxidized; a voltmeter between the two showed quite a difference of potential. Evidently the concrete acted as an electrolyte and formed a primary battery between the reinforcement and the zinc adherent to the concrete. The steel having the oxygen collected on it, acted as the positive side of a primary battery.

THE INSULATION OF CONCRETE, ESPECIALLY BELOW GRADE.

By Edward W. DeKnight.*

In 1904, and in a paper read before this Association in 1907, the following statement was made:

> 1. That steel imbedded in concrete is not safe from moisture—particularly below grade.
>
> 2. That the concrete *per se* should be protected and insulated.
>
> 3. That the security and life of the steel skeleton of the modern structure depends upon how far the columns supporting the structure are, at their bases, rotting from moisture or electrolysis.
>
> 4. That rust formation is an explosive force even greater than freezing water. That when water reaches the steel, chemical action ensues, heat is generated through decomposition or corrosion; the pent-up gas (liberated hydrogen) escapes, by bursting off a brown, volcanic cone, called rust. The immediate effect is to destroy the bond between the steel and the concrete, the heat and gas from decomposition pressing the cement away from the steel. That there then no longer exists, in fact, reinforced concrete, but the very opposite, and that which may eventually end in a collapse.

These statements seem to have caused considerable comment, at the time, possibly because of the then general lack of knowledge of the subject. Immediately thereafter, it came to me through various sources and from different parts of the country that I was an alarmist, was injuring the cement industry, etc. However, the experiments and tests made since then by many disinterested investigators and experts, substantiate the claims we made in 1904. Instead of then injuring, we were, in fact, aiding the cement

* Manager, The Hydrex Felt and Engineering Company, New York, N. Y.

industry, by enhancing the usefulness and value of concrete. In the last six years there have come on the market practically all of the many washes, stains, compounds, coatings, cement plasters and other preparations for the avowed purpose of improving the imperviousness of concrete.

Any practice, based on a wrong principle, must eventually fail. If there be anything weak or imperfect in cement construction, we want to know it and not hide it. If, for example, concrete of itself, is not a perfect protection for steel against moisture and electrolysis, especially below grade, the sooner we know it the better, so as to devise a remedy.

There is no need of confusing ourselves in a labyrinth of statistics or technical detail. There are certain elementary, self-evident truths which stand out strongly and clearly, and do not need the elucidation of any laboratory test. We know that moisture rusts steel (as now made). It is not necessary, for the second, to inquire why. We also know that if steel be kept dry, it will not decay. We think that imbedding steel in concrete, especially below grade, sufficiently protects the steel from moisture and keeps it dry, but do not actually know that it does. (We are not speaking of exceptional cases, but of the general rule.) We do actually know that if moisture reaches the steel, through the concrete itself, or a weak zone or a hair crack therein, it will not only cause the steel to corrode, but expose the steel to electrolysis, which will destroy it.

The general practice is not, as a first principle, to protect steel against moisture and electrical corrosion, which we know is vital to the steel, but to depend for protection on something which we think is, but know is not a perfect protection for the steel. Is this logical procedure? Would it not be wiser to, as a first principle, protect the steel against moisture because we know moisture to be dangerous to it, whereas we do not know that concrete is, invariably, a perfect protection? If this reasoning be sound, is it not an inevitable conclusion that the concrete, itself, should be so protected and insulated that moisture and electric currents cannot come in contact with the concrete and pass through to the steel?

In the early days we were met with the statement that concrete was watertight—then, later, that it could be made rich enough to be made watertight; or, that more steel could be added

to prevent it cracking. Later on, that washes, coatings, and cement plasters could be applied to the surface of the concrete to make it impervious. And further on, that certain compounds could be added to the concrete in the mixing to make it watertight. But it must be carefully noted that none of these methods and materials insulate the concrete.

The concrete itself needs insulation, because even assuming that such treatments will make the concrete impervious, they cannot prevent it from cracking. Even a hair crack or a weak zone in the concrete will nullify the entire value of the assumed watertightness.

We, therefore, again revert to the hypothetical question,—we think but we do not actually know that these treatments make impervious concrete, particularly below grade where it is buried and unseen, yet we do know that they all come short of the one chief thing, *i. e.,* insulation. We again claim, therefore, that we should first devise a perfect protection against moisture itself, so that it will not reach the concrete, because if it does not reach the concrete, it will not reach the steel (especially below grade).

There is some confusion in the general mind regarding protecting concrete against moisture, because of losing sight of the fact that there is

Atmospheric corrosion
Below-grade corrosion
Electrolytic corrosion.

The most serious of these is electrolytic corrosion, which, in great respect, has entirely changed the general view regarding the making of impervious concrete.

This problem was not given serious consideration in the early days of reinforced concrete construction, and its solution seems to lie on waterproofing and insulation.

Below-grade corrosion (caused, we may say, by the formation of rust simply through moisture without the effect of electric currents) was, and always has been, a serious problem. But, as above stated, it is practically only in recent years that corrosion, superinduced, as it were, by electrolytic corrosion, has caused serious anxiety and thought.

Believing that concrete is not an efficient protection for steel, especially below grade, and against electrolytic corrosion, and that

the concrete should be protected and insulated by something apart from and independent of it, in 1904 the following recommendations were made:

1. That the foundation of every steel structure should be set in practically a waterproof box.

2. That the concrete footing, under each steel column, should be protected and insulated against moisture and electrolysis.

3. That this insulation should consist of an elastic, impervious, non-conducting membrane placed on the exterior surface of the concrete.

4. That said membrane should be composed of alternate layers of bitumen compounds and fabrics—preferably an impervious felt, properly saturated and coated to withstand the injurious action of water and all below-grade conditions.

5. That this impervious non-conducting stratum or membrane should be so applied as to be practically independent of the concrete surface protected and insulated, so that the concrete could take its final conformation,—or settle, contract, expand or crack without injuring the non-conducting insulating membrane.

6. That if this theory be the correct one, it was then within the domain of the chemist and the manufacturer to make materials best adapted to the practical application of that theory.

Tests since the above theories were advanced conclusively show that a membrane of the nature above described is a perfect non-conductor against electric currents; so that if even the membrane be placed on the surface of steel, without even the use of concrete, said membrane would act as a positive protection and insulation for the steel against both moisture and electrolysis.

A sample of five layers of properly saturated and coated felt cemented together with an elastic bitumen compound making a membrane or laminated sheet about 7/16 in. thick, was tested to determine its insulation resistance to electrolytic action, *i. e.*, when subjected to a difference of electric potential in the presence of moisture. Hard rubber rings were fastened to this membrane or

made-up sheet, thus forming a watertight cup for the water. The area of the surface exposed to action was 4.2 sq. ins. The electrolyte was in all cases hydrant water with small quantities of KCl and NaCl added and the electrolyzing voltage was 5 volts. The insulation resistance of the sample was measured initially and periodically thereafter by the high sensibility, series galvanometer method, using a potential difference of 6 volts and a period of electrification of one minute. The results obtained are given in the following table:

Duration of Test. Hours.	Insulation Resistance (Meghoms).
0	Very High*
0.5	"
17.0	"
17.5	"
42.0	"
90.0	"
282.0	"
834.0	200,000†

No readable deflection could be obtained.

After the final insulation resistance reading, the sample was subjected to a high potential test by arcing around sample at about 35,000 volts. The sample was not punctured or injured, and retained its full insulation resistance. The frequency of the test voltage was 60 cycles per second, and the electrodes were needle points.

From the above test it will be readily seen that the membrane was between the water and the needle point on the other side. There was not even the intervention of concrete. The object was to make the test as severe as possible without the intervention of concrete, and to bring the needle point as near the membrane itself as possible. With the intervention of concrete, the needle point would have been some inches removed from the surface of the non-conducting membrane. Of course, imbedding the needle in concrete and then placing it in water and subjecting it to the same condi-

* The galvanometer used had a sensibility of 200 megohms per centimeter deflection at a potential difference of one volt, hence the resistance was at least 200,00 megohms.

† In all measurements excepting the last the voltage was 6,0 volts, while in the last measurement it was 100 volts.

tions and tests would, without the interposition of the non-conducting and impervious membrane, have in time resulted in the destruction of the needle through the transmission of electric current to it through concrete.

It will be noted that, as stated above and in the references* submitted, it is clearly shown that concrete, especially below grade, is not a sufficient safeguard for the imbedded steel against moisture and corrosion, that few, if any, recommendations are made as to how the concrete, itself, is to be protected. The chief aim seems to have been to devise a protection for the steel, itself, by the use of paints and other materials, or, through the making of better steel, along the line of the suggestions of Mr. Allerton S. Cushman.

It would seem proper to insulate and protect the concrete, itself, by a pliable, impervious, non-conducting membrane, composed of alternate layers of special felt—saturated and coated—, and bitumen compound, independent of the surface on which it is placed, so as to prevent moisture from coming in contact with the concrete, and thus prevent the transmission of electric currents through the concrete to the steel.

* *Proceedings,* Am. Soc. T. M., Vol. X, p. 162.

Engineering News: April 24 and November 21, 1902.—May 14, 1908.—Editorials.

Journal, Assn. of Eng. Soc., February, 1909.—Electrolysis of Concrete.

Engineering Record, July 30, 1910.—Corrosion of Iron Imbedded in Concrete.

THE WATERPROOFING OF TUNNELS.

By A. H. Harrison.*

While the comparisons of various methods of waterproofing elucidated in this discussion may apply to a greater or less extent to all classes of concrete construction, the purpose of this paper is simply to compare them in so far as they are applied to tunnel construction, *i. e.*, underground passages built without disturbing the over-lying materials. There is no doubt but that the ideal concrete structure is one in which the entire mass or body of the concrete is in itself watertight. Although the early users of Portland cement believed with it could be produced waterproof masonry, it was found to be far from as watertight as desired in modern engineering works. The early failure in the effort to produce waterproof concrete led to the adoption of the practice of surrounding the masonry with a waterproof membrane of some sort to produce the desired result. Further study of the subject convinced concrete engineers that in theory waterproof concrete was possible, and in practice might be obtained by the perfect grading and mixing of the materials forming the concrete. Concrete engineers, while constantly striving to produce a more nearly perfect concrete, realize that such is impossible without their personal inspection and attention to the grading and mixing of the constituent parts of the concrete. This inspection, however, is practically impossible, due to the fact that nearly all concrete work of any magnitude is done by an outside and disinterested contractor. In the past few years, however, it has been discovered that these difficulties may be largely overcome by the addition to the concrete of some extra substance, such as a waterproof compound, to offset such conditions.

The use of Portland cement has grown so rapidly in the past twenty years that the problem of making it impervious to water has hardly kept pace therewith. While several admirable methods of making concrete impervious to water are generally known and

* Harrison Waterproof Materials Company, New York, N. Y.

used, no one method or product is the best to use on every and all classes of concrete construction.

METHODS.

Three well-known methods of waterproofing are open to the engineer's judgment and discretion in designing tunnel construction. These methods may be defined as:

1. The Membrane Method.
2. The Internal Plaster Method.
3. The Integral Method.

The following is a brief and impartial discussion of each method as to mode of application, effectiveness of result and that which is most important, comparative cost.

1. The *Membrane Method* consists of surrounding the tunnel with an envelope of several layers of felt or woven fabric cemented together with coal tar pitch or asphalt, or by a course of brick masonry having the joints filled with a bituminous mastic. Due to the rough nature of the rock lining, it is impossible to apply the membrane directly to the sides of the tunnel. The sides, therefore, must first be smoothed out with an extra course of concrete or brick masonry. This masonry must necessarily vary in thickness according to the rock contour of the surface, but is usually designed to be 6 ins. These walls add absolutely nothing to the permanent strength of the structure, although the walls themselves, and the extra excavation necessary for same, add a considerable sum to the cost.

To these walls are applied several alternate layers of pitch and fabric, the number of layers depending upon the hydrostatic head of water. In applying the membrane, the pitch must be heated, the fumes of which are very objectionable in the confined space of a driven tunnel. The pitch must be of proper temperature, which makes it difficult of application in cold weather as it chills too quickly. Adequate drainage is necessary as it is difficult to apply the membrane against wet walls. The application to be successful must be made by skilled and conscientious labor. Joints must be carefully overlapped, as any break in the continuity of the work would be fatal. Inadequate working room

in the narrow confines of a tunnel and the fact that it is often necessary to apply the membrane in very small pieces or patches, as in the case of the Pennsylvania Railroad tunnel where the operation was similar to that of placing shingles on the roof of a house, tend to make this difficult.

Assuming that the application has been successfully made, there must yet be guarded against the danger of a puncture in the membrane while the concrete lining is being placed. Once the walls are placed, the membrane is inaccessible and that if a leak should occur, it cannot be repaired without tearing out the walls, also the membrane is now subject to the action of the liquids and gases in the surrounding soil and while on the one hand may last a lifetime, it will on the other hand rapidly disintegrate and become worthless if there is any gas drip or organic matter in the vicinity.

Another point is that the source of a leak is very often hard to trace, as for example a leak that occurred in the Hudson and Manhattan Railroad Company subway about a year ago. A leak developed in the East side of Twenty-third Street Station, the water running down over the platform onto the tracks. The leak ceased about one week later after the breaking of a 24-in. water main on the West side of Sixth Avenue at Twenty-fourth Street. This proved that the water main leaked before breaking, and that the water flowed across the subway and down the East side to a point about 250 ft. south of the source, there finding a weak point in the membrane. Many dollars were spent in tearing up the street, examining house connections, etc., in a fruitless endeavor to trace the source.

Every pound of coal burned in the power house to generate electricity is dissipated in the tunnel in the form of heat. The membrane acts as an insulator of this heat preventing any escape through the walls. While this fact may not be generally known, it is nevertheless recognized by authorities as the cause of a great deal of the discomfort in the present New York subways in warm weather. In direct comparison to this, is the comparative coolness of the Hudson and Manhattan Railroad Company tunnels in those sections not surrounded by a membranous envelope.

2. The *Plaster Method* consists in applying to the internal

tunnel walls, a waterproof cement mortar. The walls are first cleaned and chipped to insure a perfect bond, the mortar being then thoroughly troweled on with a thickness of 5/8 to 1 in. This work is generally done in two coats, scratch and finish, there usually being an interval of 24 hrs. between applications. If the walls are leaking it is necessary to drain the water off by "bleeding". Its chief objections are the high cost, the extreme difficulty of bonding new work to old, large patches of the plaster being frequently forced off the walls and the fact that it leaves the entire mass of the concrete water soaked, which is wrong on principle. If properly applied, it is very efficient.

3. The *Integral Method* consists in adding some extra substance to the concrete. The use of a water repellent compound only will be considered in this discussion, as a material of this nature possesses all the good points of other agents, plus the water repellent quality. The compound is intimately diffused throughout the concrete with the object of making the entire mass or body of the concrete watertight. Concrete has a capillary attraction for moisture. The addition of a water repellent compound breaks up the continuity of the capillaries and changes the capillary positive in concrete for moisture to negative. To obtain good results by this method, it is necessary that care be exercised in the mixing and proportioning of the materials forming the concrete. The addition of a compound will not make good defects in mixing or carelessness in use. These compounds are mostly composed of the same material as the principal element in the cement with which it combines and will not injure the permanent strength of the concrete.

The method adopted by the Hudson and Manhattan Railroad Company in the use of such a compound was as follows: Concrete was mixed in a batch mixer in two bag batches. 2 lbs. of the compound were placed in a pocket in the first two wheelbarrows of sand of each batch and dumped into the mixer. All the materials forming the concrete were turned several times before adding water to insure the thorough diffusion of the compounds.

Points in favor of this method are that it requires no extra labor for mixing or application, it needs no protective masonry coating as in the case of the membrane method, and the structure

is finished ready for use and requires no delay in order to apply waterproofing on the interior as in the case of the plaster method. The entire structure is the same and if a part is chipped away the remainder is as waterproof as before. In the use of this method on a large scale some leaks may occur, due to open joints or defective concrete. These leaks, however, can be readily patched with the same material.

Comparative Costs.

The comparative cost of the three methods can best be obtained from the cost data of the Hudson and Manhattan Railroad Company. All three methods were used on various sections of this work in New York and Jersey City. The cost of the membrane method for 6 in. concrete protection walls, and 4-ply fabric, exclusive of extra excavation and forms was $29.73 per running foot of standard two track tunnel. The plaster method was nearly as high, being $20.50 per running foot. The integral method cost $3.60 per running foot. Approximately 10,000 running feet of tunnel were waterproofed by the latter method, showing a total saving of $263,000 compared with the membrane method and $169,000 compared with the plaster method.

A further indication of the economy of the integral method may be gained by a study of the cost of waterproofing Routes 5, 9, and 20 of the Triborough subway according to the specifications of the Public Service Commission, based on figures of the various low bidders. The total of the low bids for waterproofing, which include protective concrete masonry, woven fabric cemented with pitch, brick laid in asphalt mastic and extra excavation for concrete masonry, amounts to the enormous sum of $3,888,892.00. This cost allows only for waterproofing the stations and below a line drawn two feet above ground water level. Notation is made on the drawings, however, that additional waterproofing may be required where designated by the engineers.

If the integral method were used, the entire subway from end to end could be made waterproof for a sum not exceeding $500,000.00, meaning a saving of $3,388,892.00 or 667 per cent. Should any leaks occur by the use of the integral method, there is ample margin left for cost of patching the same.

WATERPROOFING WITH WATER.

By Cloyd M. Chapman.*

It is of the first and greatest importance in the making of a waterproof concrete that the mixture shall be so proportioned as to sizes of the particles that the mass shall be of maximum density and therefore of minimum voids. Yet, after the utmost diligence has been employed in the selection of the mixture and in the proportioning of it, there are other factors still to be considered, one of which, very often not considered, is the amount of water to be used.

It has long been recognized that the quantity of water used in mixing concrete had a large influence upon the impermeability of the resulting mass. This fact has been well established and most works on the subject of concrete, however brief, call attention to it. That impermeability may be regulated to some extent by the amount of water used is of great importance to the concrete industry. It is perhaps of as great importance to the block manufacturers as to any other branch of the industry. We frequently read the claims of the block machine manufacturers to the effect that this or that machine produces an impervious block because it is made "wet" or "semi-wet" or "dry". It is not generally realized what a large difference in permeability a small difference in the percentage of water used will make. The theory is generally held that other conditions being the same, a wet mixture makes a more waterproof concrete and that a dry mixture produces a more porous mass unless very tightly compacted by much hard tamping or enormous pressure.

It was in an endeavor to make a very spongy porous concrete for a special purpose that Westinghouse, Church, Kerr and Company made up a series of blocks with varying percentages of water and noted the great difference in porosity made by slight changes in the percentage of water used. The blocks used in this series of experiments were about 3½ ins. square and 2 ins. high, with a

* Engineer in Charge, Westinghouse, Church, Kerr and Company, New York, N. Y.

circular depression or cup in the top about 2½ ins. in diameter and 1 in. deep. Some of these blocks were made of 1:2 and 1:3 mixtures of Portland cement and a good grade of building sand, while others were made of the same mixtures of cement and crushed granite screened through a 5-mesh sieve.

The molds were of wood and the mixtures were tamped in by hand with a light tamper. A metal bell was used to form the depression or cup. In some cases the blocks were allowed to set

FIG. 1.—TYPE OF BLOCKS, MOLDS AND APPARATUS USED IN TEST.

in a damp closet for 24 hours before being exposed to the air, while in other cases the blocks were left in the air to set as soon as made. The day after the blocks were made the inside surface of the depression was brushed with a wire brush to remove any cement film or laitance from the surface, which might affect the results. When the blocks were 1 week old 30 cu. cm. of water were placed in the depression of each block and the time noted which was required for the block to absorb all the water. Fig. 1 shows a few of the blocks under test, the type of molds in which they were made and the general method of molding them.

After the first test the blocks were allowed to dry out and were again tested at 2 weeks old. After the second test they were exposed to the weather on the roof and tested again at the end of a month. Many blocks were made of each mixture of materials and of each percentage of water, and the average time taken in absorbing the water was used in comparing results. In all about 230 blocks were made and tested.

Fig. 2 shows graphically the results obtained at the first test when the blocks were 1 week old. The upper curve represents the

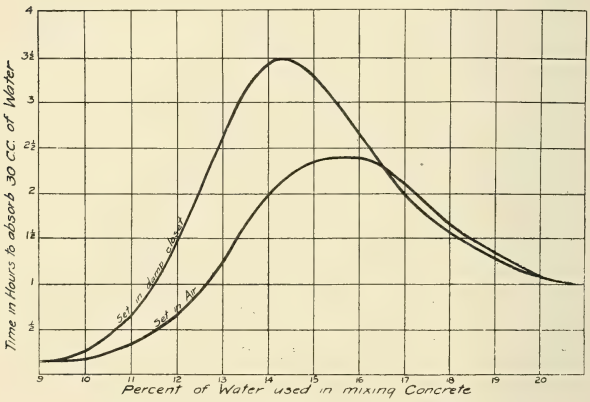

FIG. 2.—DIAGRAM SHOWING RATE OF ABSORPTION.

results obtained on the blocks which were placed for 24 hours in a damp closet before being exposed to the air, and the lower curve shows results on blocks dried in air immediately after making and not sprinkled. The ordinates represent the time in hours required for the block to absorb 30 cu. cm. of water placed in the depression, while the abscissae represent the amount of water used in percentage of the total weight of cement and aggregates. It will be noted that the results obtained with blocks which set in the damp closet were considerably better than those obtained with the blocks which set in dry air. There was so little difference between the results at the end of 1 week and those at the

end of 2 weeks and 1 month that only the results at the age of 1 week are shown. The other curves would practically coincide with these. While the best percentage of water probably varies with different cements and aggregates and should be determined for any particular combination that is to be used, yet it is evident that for the cement and sand used in these experiments the amount of water should lie between 13 and 16 per cent. of the total mixture by weight, if the blocks are to be cured in a damp place. When exposed to the air immediately after making, somewhat more water should be used—say, between 14 and 17 per cent. The

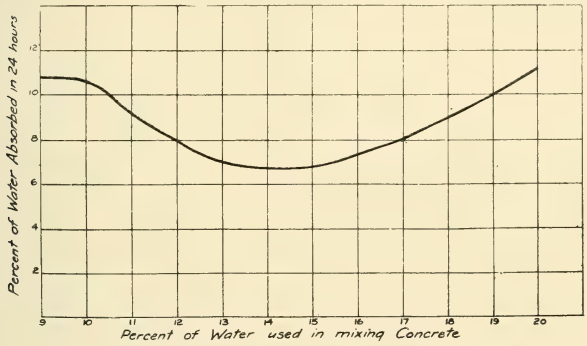

FIG. 3.—DIAGRAM SHOWING AMOUNT OF ABSORPTION.

point it is desired to emphasize is the importance of using the right amount of water. It cannot be stated that under other conditions the percentages here given will be correct.

The above experiments indicate, not the amount of absorption of the blocks, but the rapidity or rate of absorption. After the blocks had their third test at the age of 30 days, they were dried out for a week in a warm room, carefully weighed and then immersed for 24 hours in water. The surplus water was then wiped from their surface and they were weighed again. Fig. 3 shows graphically the results of this series of absorption tests. In this curve the ordinates represent the percentage of water, by weight, absorbed by the dry blocks during 24 hours immersion, while the

abscissae represent the percentage of water, by weight, used in mixing the concrete. The relation to the former curves is at once apparent. The least absorption occurs with about 14 per cent. of water and it is fairly uniform between 13 and 15 per cent. By either method of determination, that is, by the rate of absorption or by amount of absorption, there is derived the same conclusion. A concrete may be too wet or too dry to produce the most impermeable product, and one extreme is as surely an evil as the other, if impermeability is desired.

DISCUSSION.

Mr. Rudolph J. Wig.—I would like to ask whether Mr. Chapman considers the quality of absorption the same as permeability, *i. e.*, is there a distinction between absorption and permeability? If the absorption of a concrete is a certain amount, will the permeability of the same concrete be in proportion; that is, will the absorption and permeability curves run parallel to one another? Mr. Wig.

Mr. Cloyd M. Chapman.—That is a question of terminology. We consider "rate of absorption" as the amount of water absorbed in a given time, and "total absorption" as the amount of water absorbed in any extended length of time. To determine the total absorption a long time must be allowed for the water to permeate. The term permeability is often applied to the amount of water which will pass entirely through the concrete; that is, pass into a wall and out on the other side. That is not synonymous with permeability as the word was used in this paper; which was, the rate at which the cube would take up water or the amount of water which it would take up. Some of our experiments indicate that while concrete will soon become wet on the outside of a cube filled with water, yet no water passes through and drips off the cube. It seems there should be a distinction between permeability meaning the amount it will take up and permeability meaning the amount of water which will pass through. We made no tests to find the amount of water which will pass through and come out on the other side, and drip off. This would probably, in the case of these dense mixtures, require pressure. Mr. Chapman.

Mr. Wig.—We made several thousand rather extensive tests. I would not say that they cannot be criticised, because they can, although they were carefully made. We found there was a decided difference between the quality of absorption and permeability. I can illustrate this very well. A relatively lean mortar made with a very fine sand will be impermeable. A 1:3, 4, 5 or even 1:6 mixture will be impermeable under a considerable head. The same mortar, however, will be very absorbent. A coarse sand in the same Mr. Wig.

Mr. Wig. mixture will show the mortar very permeable, but the absorption will be low. In the first instance there are many pores in the mortar, but they are not connected pores. In the second case there are fewer pores but the larger surfaces of the particles of sand offer connected passages along these surfaces.

The addition of foreign material, grading the sand, etc., affected these qualities differently as the mortar aged. The rates of absorption were found to decrease materially with age and the permeability decreased many times as fast; that is, concrete which possibly would allow water to penetrate it when 2 or 3 weeks old, after aging several weeks longer would be entirely impermeable. We tested some lean mortars, exposing them to pressures as high as 80 lbs. per sq. in., and the water did not penetrate at any time during the 2 weeks' exposure. I think we should try to separate the terms permeability or waterproofing and absorption. Absorption is an entirely different quality from permeability.

Mr. Larned. MR. E. S. LARNED.—Was a different percentage of water used in mixing the 1:2 and 1:3 mortars?

Mr. Chapman. MR. CHAPMAN.—From 9 to 20 per cent. of gauge water was used with the 1:2 and 1:3 mixture. The results of both varied, but were slightly better with the 1:2 than with the 1:3 mixture. The figures used in plotting the curves were the average of both.

Mr. Larned. MR. LARNED.—In leaner proportions than 1:3 would you not deem it expedient to use more water?

Mr. Chapman. MR. CHAPMAN.—As stated in the paper, these percentages of water were not given with the idea that they would be used in the field. The paper was simply to point out that there are certain percentages of water which are better in certain cases than other percentages, and that in each particular case the percentage should be determined. With other materials and other cements this curve might not apply, but the shape of the curve would probably be approximately the same. That is, it would be a curve that would show that there are definite points at which the permeability is low.

Mr. Larned. MR. LARNED.—It has been suggested to me that the fineness of cement has a very important influence upon the amount of water to be used and that the finer the cement the less water can be used successfully in the concrete. It seems to require a relatively drier mix. I would like to ask if some one had made research or formed any conclusions in regard to that.

Mr. Harry A. Taylor.—I have had a little experience in the use of waterproofing, not from a chemical or chemist's point of view but from its use in cement blocks and other products. I find the so called dry waterproofing compounds a detriment rather than a benefit to the man who makes cement blocks, for the very reason that you cannot get enough water into the block so that it will crystallize and make a hard block with the same strength as a block without waterproofing. Mr. Taylor.

Using waterproofing, I have had my failures in making cement blocks. Now I am making cement blocks without waterproofing and water stands on top of them and will not go through. I believe it evaporates rather than sinks into the block.

I do not claim to use more cement in making blocks than other people, and yet I obtain a harder block, a block with greater crushing strength than those who are using as much cement in their concrete as I am. The reason I obtain a hard block is just simply that the voids are filled with water until the cement crystallizes, and that is my theory. If sufficient water is supplied to fill the voids and keep them filled until crystallization is finished, a concrete results that is almost absolutely waterproof.

Mr. Myron H. Lewis.—In the last week or so, I was surprised to hear from a number of engineers that they thought waterproofing unnecessary, and the discussion has brought out some more of the same opinions. I think that Mr. Spackman's remarks and those on the difference in results between laboratory and practical tests bear out very strongly what I have to say. Mr. Lewis.

While engineers often tell us that no waterproofing is necessary in concrete structures, the opinion is prevalent among the public at large, that concrete buildings are damp, therefore, unwholesome. However, it is possible to so grade and blend the materials of the concrete as to obtain a watertight product. But the consumer of concrete knows that while watertight concrete is possible, there are a number of examples of leaky structures due to one cause or another. Those who argue that waterproofing is not necessary usually base their conclusion on their own experience or rather capabilities of using the most careful methods, with strict supervision of every detail of proportioning, mixing and laying of

Mr. Lewis. the concrete, and the results obtained under such conditions are model and are the best obtainable both as to density and impermeability.* But for every one such perfect job, others turn out anything, but model, and to maintain that waterproofing is not necessary because one job turns out tight, is certainly not a justifiable position for the concrete men to take. In making this statement I am not speaking from hearsay but as an editor and a contributing editor to a number of concrete journals. I inserted several invitations requesting readers to advise me if they were troubles with leaky structures. While I expected some affirmative replies, I was certainly astonished at the large number received and they were limited to no section of the country, North, East, South and West alike sharing the trouble of leaky concrete.

We are told that waterproofing is not necessary, and even our highly technical societies have spread the belief that no direct remedy is obtained by employing waterproofing compounds. I was glad to hear Mr. Spackman agree on the general proposition that waterproofing is a necessity in many conditions to be found in concrete work.

While admitting that it is possible to secure waterproof concrete by proper proportioning and handling of the material, the large number of leaky structures throughout the country make waterproofing of structures advisable as a matter of precaution and insurance, particularly in structures below ground water or tidewater level.

I had the pleasure of listening to Prof. Norton's paper on the effect of heat on concrete in which he closed by calling attention to the frightful annual fire loss in the United States. It would be a difficult matter to estimate the tremendous annual loss caused by

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* In this connection the following paragraph, from the report of the Committee on Treatment of Concrete Surfaces, may be of interest.—ED.

The Committee has made some further study of the waterproofing compounds on the market which are to be incorporated in the concrete at the time of mixing and is of the opinion that in the results obtained with this class of materials, workmanship counts for more than the compounds themselves. Whether the waterproofing ingredients be collodial clay, lime or a patented powder, paste or liquid, it must be most thoroughly and uniformly mixed throughout the mass of the concrete, and this can only be done well in a batch mixer and by leaving the charge in it an extra length of time. With carefully selected and graded materials, thoroughly mixed and skillfully placed, concrete is in itself waterproof without the addition of any foreign matter.

discoloration, disfigurement and unsanitary and other objectionable conditions due to the injurious action of water on concrete and stone structures and structural materials. But just because water works slowly it works none the less surely, and I believe that the use of any method or material, which will retard even for a limited period the injurious action of the water, is justified on the basis of insurance. Mr. Lewis.

I wish to call special attention to the question of waterproofing sewers. This question has received hardly any consideration at all from sewer engineers, although it is one of the greatest importance. Dr. George M. Price, sanitary inspector for many years of the New York Tenement House Department, stated that the soil of our city was like a graveyard from the leakage of refuse through sewer lining, and at the same time he pointed out the unsanitary conditions due to leaks of buildings as found in 20 per cent. of the New York tenements.

Mr. George T. Hammond,* designing engineer, of the sewer department of Brooklyn, New York, has collected some valuable facts on the infiltration of ground water into sewers and has said that municipalities are wasting princely sums in pumping and treating such ground water at purification and disposal plants. The following are his figures on the leakage of ground water into sewers in various cities:

New Orleans, 1,250,000 gal. per sq. mile.
Columbus, O., 100 to 300 per cent. of dry weather flow.
Kalamazoo, Mich., 20 per cent. of capacity.
Norfolk, Va., 60 per cent. of pumping.
Canton, O., 70,000 gal. per mile.
Brockton, Mass., 2,500 gal. per mile.
E. Orange, N. J., 110 per cent. of dry weather flow.

They show conclusively the possibility of lower maintenance costs if the sewers were properly waterproofed.

It is generally agreed upon that watertight concrete is the only solution of prevention of disintegration of concrete by sewage, sewer gases, alkali and other corrosive chemicals.

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* See discussion by Mr. Hammond, p. 690.—Ed.

Mr. Lewis. There are many virtues to the successful methods of waterproofing on the market and there are no cure-alls. The whole secret of successful waterproofing lies in selecting the method and material suitable for the conditions at hand. Many of the standard materials will give good results if intelligently handled. All modern methods of waterproofing are outlined in Table I, as well as the special features of each method so that a selection may be made most suitable for the conditions to be overcome.

There are many cases where neither the integral nor the membrane method will meet the conditions. Take, for instance, the preservation of beautiful stone monuments or decorative stone in process of decay, the disfigurement of buildings by efflorescence—which is an eyesore in many parts of this city. In no less a place than the Columbia University Chapel, surrounded by scions of engineering and architecture, a most magnificent brick building is disfigured by an ugly white stain marring its otherwise perfect and beautiful appearance. Almost anywhere in this city, close observation of stone masonry will disclose chipped and spalled arrises, sills and copings due to the absorption of water and the consequent freezing and thawing. In a recent discussion, a number of engineers present, described various troubles they were involved in by neglecting to take the precaution of waterproofing their concrete.

These observations may be continued indefinitely and sufficient evidence can be produced to convince the most skeptical that waterproofing of structures is a precaution which should always be provided for in any concrete structure intended for habitancy and where leakage is in any way objectionable.*

I have often been asked the question as to which was the best waterproofing material. It is a question which cannot be answered unless all of the conditions of the work are described, and it is for partially answering this question that Table I was prepared.

There are so many conditions to be considered, that I cannot

* The author of these remarks uses the word "waterproofing" indiscriminately as applied to materials and processes. Waterproofing, in the strict sense, has to do with the design of the structure, so as to render it waterproof. This may be accomplished without the use of special waterproofing compounds or materials.—ED.

agree with the statement by the Committee of this Association, and that of the American Society for Testing Materials, that waterproofing materials have been found of doubtful value. Many cases have not been covered by the committee work and broad statements of condemnation are misleading. Buildings which have been erected for a number of years are often found leaky and damp. These must be made tight and waterproofing materials must be employed for the purpose and when a broad statement is made that all such materials are of doubtful value, the consumer is induced to believe that his difficulties cannot be remedied, and that is certainly not so. Mr. Lewis.

A great many tests have been made during the last few years on various waterproofing materials. It is of great importance to know the results of these tests, but in addition to these results, we must also get in touch with the consumer, and obtain information from the school of practical experience, as well, before we can pass final judgment as to the adequacy or inadequacy of such materials.

Much good work has been done by the waterproofing concerns and I do not think that we are justified in condemning the industry to the extent that it has been condemned, because I do not think we have enough information from the user of concrete.

Before closing, I wish to say a word on the non-absorbent and the permeability question. It was that very question which led me to believe in water repellent compounds which are added to cement, either dry or in liquid state. Mr. Chapman pointed out that almost all of his blocks absorbed some water. This absorption must be avoided in a concrete building, in which decorations or plaster is placed directly on the interior surface of the concrete, for such absorption, unless checked, will ultimately result in the destruction of the decoration. The addition of water repellent materials turns an absorbent concrete into a non-absorbent concrete or what I have termed from a capillary positive material into a capillary negative material. In other words, if this watershedding material will last sufficiently long to keep out the water three or four years, I think we are justified in using it.

In closing these remarks, I have a few questions to ask which I hope will come up during the discussion and on which we may be able to obtain some light.

TABLE I.—OUTLINE OF METHODS AVAILABLE FOR VARIOUS STRUCTURES.

	Nature of Structure.	During Construction (New Work).	Remarks.	After Construction (Old Work).	Remarks.
BUILDINGS.	Exposed walls.	D	For damp proofing only.	B, C	D — Principally for brick and stone walls; not advisable on smooth concrete walls.
		B, C	For damp proofing and preservation of stone.		
	Super-structural floors	I, K, J, G, M	For bath rooms and floors liable to be wet.	G, M	
	Foundation pits.	I, J	For resisting dampness only.	G	For dampness only or pressure work.
	Foundation trenches.	I, J	"	G	
	Foundation footings, walls and floors.	I, J	"	G	
	Walls, reinforced	G, K, L, M	For resisting water pressure.	G, M	For resisting water pressure.
	Walls, not reinforced.	K, L, M	"	G	"
	Floors, reinforced.	G, K, L, M	"	G	"
	Floors, not reinforced.	K, L, M	"	G, M	"
	Column footings, heavily loaded.	O			
	Concrete block, etc.	G, B, C		B, C	
	Preservation of stone.	B, C	Also for preventing efflorescence.	B, C	
RAILROADS.	Subways and tunnels.	K, L, M, G		G, H	
	Bridges.	N, K, L, M		G, M	
	Culverts.	E, F		G, A, B	
	Retaining walls.	E, F, K, L		G, A, B, H	
	Arches.	E, F, K, L, M		G, A, B, H	
WATER SUPPLY AND SEWERAGE.	Reservoir banks.	K, L, M		K, L, M	
	Dams.	E, F, H			
	Conduits.	E, F, G		A, B	
	Tanks.	G, F, B		B, A	
	Chambers.	G, F, B, A		G, B, A	
	Sewers.	E, F, K			
	Manholes.	F, G		G, H	

MODERN WATERPROOFING PROCESSES.

INDEX AND DESCRIPTION OF METHODS.

Method.	Classification.	Material Employed.	Manner of Application.	Position in Structure.	Thickness, etc.	Remarks.
A	Surface coating.	Solutions of alum and soap (Sylvester process).	As a paint.	On exterior surface.	4 to 7 coats.	Applicable to new or old work; not to underground work.
B	"	Paraffine in saturated solution.	"	"	2 to 3 coats.	Applicable to new or old work; not to underground work. Also a preservative for building stone.
C	"	Melted paraffine wax applied hot.	"	On exterior surface (surfaces previously heated).	Penetrates $\frac{1}{8}$ to $\frac{1}{2}$ inch.	Applicable to new or old work; not to underground work. Also a preservative for building stone.
D	"	Proprietary bituminous compounds, applied cold.	"	On interior surface.	2 coats.	Used also as a substitute for furring and lathing.
E	Integral or rigid.	Masonry or concrete correctly proportioned.				Available only during construction.
F	"	Masonry or concrete correctly proportioned. With cement waterproofing compound.	In the body of the work.			Available only during construction.
G	"	Cement mortar with compound.	As a plaster or stucco.	On interior surface.	1 to 2 ins. on floors. $\frac{1}{4}$ to $\frac{5}{8}$ in. on walls.	For new work or remedying defective work.
H	"	Cement mortar or bituminous grout.	Pumped under pressure	In interior of mass.		Employed principally to fill voids otherwise inaccessible.
I	Membrane or elastic.	Asphalt or coal tar pitch with felt or burlap.	In alternate layers.	On surfaces exposed to water.	1 to 3 ply.	Applicable only during construction for subsurface work. To resist dampness, increase thickness according to pressure.
J	"	Proprietary bituminous compounds, applied cold with felt or burlap.	"	"	"	
K	"	Coal tar pitch, applied hot with felt.	"	"	2 to 10 ply.	For water pressure work.
L	"	Asphalt applied hot with felt or special fabrics.	"	"	2 to 8 ply.	Asphalt not to be used in ground polluted by gas drip, etc.
M	"	Asphalt mastic or	As plaster or coating.	"	$\frac{1}{4}$ in. to 1 in.	
N	"	Asphalt mastic alternating with saturated felt layers.		"	$\frac{1}{4}$ in. to 1 in. mastic. 2 to 5 ply felt.	Useful particularly for bridge waterproofing subject to vibration.
O		Sheet lead or copper.		Under column footings.		Used as substitute for felt when column loads are very heavy.

Mr. Lewis. 1. Has any one in the Association made tests to determine the effect of the action of sewage or sewer gases on waterproofing materials?

This question has been asked me a number of time and I have tried to get the opinions of the leading chemists on the subject. While they give opinions, there is nothing definite to base those opinions on. I would like to see some tests made by suspending asphaltic and coal tar felts, and other compounds in sewage and determine how long they will last under such conditions.

2. I wish to ask how long will the metallic stearates last in exposed locations? These are the bases of a number of waterproofing materials. Those who have used them and found them to have a very short life ought to tell us. If they have not failed, I think there is in such places as tunnel linings a very large field of use.

3. I would like to know what is the longest time, not only as determined by laboratory tests, but from the use in actual work, that surface coatings have succeeded in holding out water. I know of cases ranging from four to ten years and in some cases very much longer. The obelisk was waterproofed over 20 years ago and is still immune from the elements.

We should not complain if surface materials do not last longer than four or five years. If we can get a life of five years out of any material exposed to the elements, we are doing mighty well, and it is unreasonable to ask for any greater permanency. The unit cost per annum is so small compared with other forms of insurance and protection that we are justified in spending money for waterproofing insurance as much as we are in spending it for fire or contingencies of other kinds.

4. I would like to ask is it fair to the large capital invested in the waterproofing industry to conclude from laboratory tests only that waterproofing materials are of doubtful value when we know that there are structures throughout the country which have been treated with these materials and in which they have not been found wanting?

I submit these questions for the purpose of bringing out facts from concrete users, men who have tried many materials and from whom we can get information which is not based only on labora-

tory experiments. The great divergence between conditions in the laboratory, and outside of it, has been well pointed out by Mr. Wig. I trust that a satisfactory explanation of the marked difference will soon be found. If waterproofing materials are of doubtful value and have but a short life, what are we going to do with the structures which prove damp and leaky. How shall we remedy such conditions when we find them? I firmly do believe that the Committees' conclusions are not justified in a broad sense. I believe that where such reports are made, the limitations should firmly and clearly be pointed out. Mr. Lewis.

The President.—The committees whose work has been somewhat questioned have had to do with the question of trying to make a bad job good; and the statement that a lot of bad structures have leaked and that compounds and other things can be used to make them usable is not relevant to the question of whether concrete can be made water repellent or watertight. Now these committees have come to the question of how to use cement or concrete so as to make it suit the conditions for which it is designed —a decidedly distinct difference between trying to get up something which does not fulfil those conditions. The President.

Now as to whether their reports have invited such discussion, I will state I am a member of the Masonry Committee of the American Railway Engineering Association, and that committee has had reports from engineers of railroads using every kind of waterproofing that is known, and the conclusion to-day is that there is no material that will waterproof; and the inevitable conclusion after reading such reports is that the best way to waterproof is with water and the use of reinforcement to distribute the cracks.

Mr. Wig.—I would like to ask Mr. Lewis, who has related so many instances where concrete is not satisfactory as a waterproofing material, if he has given the same study and the same investigation to the failure of waterproofing compounds in the field, or if he finds that they do not fail in the field, or if they do fail, if he has classified them so, which classes may be recommended as most dependable. Mr. Wig.

Mr. Lewis.—I wish to say that I have endeavored to obtain from the cement user as much information on that particular point as I possibly could. I do not care to go into the names of com- Mr. Lewis

Mr. Lewis. pounds; it would look like advertising trade products, and I have no direct or indirect interest in any of them. I know, however, that materials of the water repellent nature have given satisfaction up to three or four years. Materials of a paraffine nature have done so for a great deal longer than that. Any one interested in such work can have a half a dozen firms in this city take contracts guaranteed under bond to produce a watertight structure by the use of materials of this kind. If a firm is willing to give a bond with a guarantee that the work will be tight for a reasonable number of years, and if there are half a dozen or more of such firms who are willing to do likewise, we can rest assured of results, particularly as they are willing to back their work with their money.

I also think the constant increase in the consumption of waterproofing materials throughout the country is very evident indication that the cement user is finding them of some actual, potent value. It could not be possible that the consumption of all kinds of waterproofing material would be increasing, if a majority of consumers were finding them of very doubtful value.

There have been any number of failures; no doubt about that. There have been just as many failures in waterproofing as in in any other line of concrete work. Men are often called in to repair work done by other people. There is no waterproofing concern in the country that can boast of not having had failures. Failures in waterproofing as well as in other lines have taught how work should not be done. Some failures are to be expected in every line of business, but a healthy business progresses in spite of them, and they help to place such business on a sounder basis.

The President. THE PRESIDENT.—The Chair would call your attention to the fact, as I think Mr. Lewis brought out some years ago, that there is such a thing as waterproofing engineering, and that the man with a structure to waterproof must study the conditions. If he does anything with a solid monolith of concrete he has a different condition to meet than a house which is made of blocks which may be watertight themselves and the joints may leak; and the same way with the basements of buildings.

The thing this Association is seriously considering is the question of whether or not you can take concrete, mix it and place it in position and get a structure that will resist either the penetra-

tion of water from the outside or is satisfactory under the conditions of dampness. The President.

In this period of transition from porous to watertight concrete, where we are lacking in skill how to make it to the condition where we can make it properly, there are doubtless some materials which will help tide over those conditions. The Chair has not made any effort to condemn waterproofing compounds, but to state that it is not necessary to use compounds if you treat the material intelligently and consider the conditions for which the structure is designed.

TOPICAL DISCUSSION ON WATERPROOFING OF SEWERS.

Mr. Hammond.

MR. GEORGE T. HAMMOND* (by letter).—My attention has been called to the discussion on the necessity for waterproofing in which has been submitted certain data collected by myself relative to the question of the infiltration of water into sewers. The data submitted is incomplete in that it does not state either the character of the sewers or the conditions affecting the same. I desire, therefore, to submit the following in further discussion of this subject.

Infiltration into sewers forms one of the most interesting and difficult of the various problems in sewerage engineering. Examination of a considerable number of sewerage systems and reference to annual reports reveals the fact that there are very few, if any, systems free from this trouble. As a rule sewers admit ground water where they pass through wet ground, and permit the leakage of sewage where they pass through dry ground. I am aware that this is a broad statement, but I believe it is justified by the facts. As to the effect of such leakage, there may be room for difference of opinion; in many cases it probably does no harm; in some instances it may even be beneficial in that it removes ground water and lowers the water table in low districts, making them more healthy. This is especially the case where infiltration occurs into storm water sewers or drains and even into combined sewers, if the leaks are not of such a character as to permit the movement of sand into the sewer. Some of the older sewerage systems were in part constructed with pipes laid with open joints, in order that the ground water might enter the sewers and the wet places be drained; but this was always bad practice. Where there are small leaks in a sewer in dry soil, which is seldom or never submerged below the water table, the silting up effect of the floating matters in sewage soon closes the leaks, and prevents the contents of the sewer from infiltrating the soil; but this silting does not occur so readily where the ground water flows sometimes into the sewer, and the sewage

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flows sometimes out of the sewer; as may be caused by the varying head of ground water. Mr. Hammond.

Leaks into the sewers are usually of the greatest interest and importance to the sewer engineer in these days; and especially concern the separate system of sewerage. In most systems of this kind, the sewage flow must be raised or pumped from a low to a higher elevation; and every pound of water so raised requires the expenditure of mechanical energy, which must be paid for in the maintenance of the plant. The results of such leaks sometimes are such that the cost of pumping is more than doubled. Infiltration may not be objectionable in storm sewers, or drains, but unquestionably it is bad in all other sewers. It is standard practice now, among sanitarians, to require that all sewers shall be so nearly water-tight that the contents will neither infiltrate the ground around the sewers, nor escape into the various channels leading into streams, nor pollute the spring water which finds its way into wells. To an extent seldom recognized, even in these days of progress in all of the sciences, the health of a community depends upon the protection of the soil from the contamination caused by sewage.

The admission of ground water into sewers, especially into sanitary sewers, provided in the separate system, where the sewage is to be pumped, and treated to remove putrescibility, is certainly improper and to be condemned. Waterproofing should be employed, if necessary, to secure tight work, and the best of structural materials should be used in all parts of the sewerage systems, whether they be disposal plant, pump well, manholes, or sewers. Concrete is undoubtedly the best material for such structures and its use is generally preferred to brickwork, which is less homogeneous and less reliable. But good or bad work can be done with any material. Careful design is rendered of no effect by careless construction.

A sewerage system includes (*a*) collectors, which are the pipe sewers laid in the various streets, and which are usually of vitrified, concrete or iron pipe; some plants using all of these material in different places, in accordance with the designer's ideas of local requirements.

(*b*) Main sewers, which receive the collectors and carry the

Mr. Hammond. sewage to the disposal plant, or to the intercepting sewers as the case may be. These sewer mains have very generally been constructed of brick, and usually have proved to be the portion of the system requiring the greatest expense, both in construction and maintenance; and they usually have permitted large quantities of ground water to enter the system.

(*c*) Main intercepting sewers, which usually must be laid at a low level, frequently in ground water, for the purpose of receiving the flow of sewers crossing them at a higher level, and of carrying the sewage to a pump well, or to a disposal plant; are often the greatest offenders in admitting ground water. Modern practice is to build them of concrete, either formed in place or made into sections of pipe and laid like pipe sewers; with the result that but little leakage occurs into them. In all such sewers laid in ground water, the structures should be waterproofed, as has been done with marked success in Baltimore.

Failures and leaks in pipe sewers are caused chiefly by the following:

1. Pipe of improper material for sewers.
2. Defective or broken pipe.
3. Defective joints.
4. Defective house connections.
5. Improper foundations.
6. Careless or improper backfilling of trenches.
7. Improperly constructed brickwork and concrete.
8. Disintegration of pipe or masonry, caused by chemicals or impurities in solution in the sewage or in the ground water, and from steam admitted to the sewers.
9. Failures from other causes not classified, as unexpected contraction and extension, impact, etc.

Some time since, the writer was called upon to prepare a discussion on the subject of ground water entering sewers, and some of the remarks that follow are repeated from that discussion.

Frequently sanitary separate sewers are so low in the ground in relation to the purification works, that pumping is required to deliver the sewage into the tanks and upon the filter beds. Here pumping costs no small sum; hence, it is in such a case doubly

important that the pipes and joints shall be tight and admit no ground water to unnecessarily increase the expense of operation. Mr. Hammond.

The importance of perfect work and first-class design, is perhaps no greater in any branch of the engineering profession than it is in sanitary works. Indeed, it may justly be observed that the public generally understands this to be so, and is alive to its importance to a remarkable extent; and yet the time is not long past when the mere mention of the name sewer was considered vulgar, if not indelicate and improper.

Julius W. Adams, the pioneer in American sanitation, tells us that in 1865, when he was commissioned to design the sewerage system for the City of Brooklyn, that few of the important problems involved in such a great work were recognized even by the engineering profession; and "so ignorant was the public generally on the subject, that in our principal cities, notably in New York, it was the custom for the residents in the different streets to petition for a specific size of sewer in front of their premises, regardless of the dimensions of the sewer from which it led or into which it was proposed to be discharged, but of whatever size a majority of the property owners chose to favor. If the latter were willing to pay for a large sewer, so much the better would be the drainage, was the supposition." Of course, in such a crude stage of the art, no one thought of the leakage into the sewers of the ground water, or of the escape of the sewage into the ground. As a consequence, the ground of the city has come to fulfil the dictum of Pettenkoffer, and has become "more contaminated than a graveyard" in some places; and no one with any regard for his life would risk drinking water from any well or spring situated within such a locality. Strangely enough it was the boast of some of the sewer builders that the construction of the sewers had lowered the ground water; and this has usually been the result of sewer construction; partly from leakage into the sewers, especially when inferior pipe with poorly made joints has been used. Observations on the sewerage systems of Brooklyn and New York show that the sewers of both cities admit a large amount of leakage. But the ground water enters not only through imperfectly or improperly made pipes or pipe-joints; it seems evident, from observations made by the writer, that a great amount of leakage also enters the sewerage system through leak-

Mr. Hammond. ing brick work, and the pores of the brick masonry employed in manholes and large outlet sewers. Where the sewers discharge directly into tidewater, such leakage is not now a matter of much consequence; but where the sewage is treated as it is in the East New York District, and at Coney Island, the matter is quite important, as it adds greatly to the cost. At the present time efforts are being made to lay all sewers with impervious joints; and the question of waterproofing is receiving careful study.

A word should be said here about the use of iron pipe in sewer construction. Of course it is clearly evident to all of us that one can get a more nearly waterproof job by the use of iron with lead or leadite joints than in any other manner; but it should never be forgotten that "a sewer of iron pipe is put down to be replaced later," and cannot be regarded as a permanent structure. Cast iron, when unprotected, is attacked by the various sewage impurities and chemical compounds quite rapidly, and its endurance depends largely upon protecting its surfaces. The life of a 16-in. pipe placed by the writer some years ago, was about seven years; and in the use of small size sewers and house soil-pipes, cases might be mentioned in which the tuberculation of the interior surface of a small iron pipe together with crusts or rusts, were sufficient in a few years to fill the lumen of the pipe to such an extent as to render the sewer ineffective as a carrier. No one can tell how long an iron pipe used for sewage will last. There doubtless are cases in which extra heavy pipe has lasted many years, especially when of sizes above 24 in. in diameter; but these instances are rather misleading and exceptional. Probably 20 to 30 years is as long as one should expect such a pipe to last, exposed to the action of sewage; and this age may be greatly reduced by causes which cannot be anticipated or provided against by the engineer at the time of construction.

For sewerage, an iron pipe should never be used if its use is avoidable; concrete, or vitrified stoneware pipe are preferable from almost every point of view, and are not subject to corrosion; and do not lose their capacity by tuberculation and rusting of the interior; furthermore, they are not endangered by electrolysis, against which it is scarcely possible to feel sure that any iron sewer pipe is safe.

Where the vitrified pipe is not of sufficient strength to with-

stand the earth pressure, or the pressure developed in ramming, special provisions, such as will readily occur to the designer, should be employed. A complete protection of concrete may in some cases be indicated, and should be used, rather than that an iron sewer should be selected. Mr. Hammond.

The waterproofing of sewers is perfectly practicable. The extent to which it should be employed, and the best method adaptable to a given case are the problems to be met, as well as the probable length of time during which the waterproofing employed will continue effective, and the best should be selected for each special case.

There are various methods here for producing the effect, as elsewhere. Coatings of bituminous materials may be applied to the external surface of the sewer, and gaskets dipped in tar or maltha or hot asphalt may be employed in the joints; or the whole interior of the sewer may, in the case of large masonry structures, be treated with a plaster, as was done in the experiments made by the U. S. Geological Survey, for irrigation force main. Again, the sewers may be designed to be built of concrete pipe rendered impervious by exterior coatings of asphalt paint or maltha; or by having an antihydrous waterproofing compound mixed with the cement used in the concrete.

Recent experiments seem to show that even the admixture with the concrete of clay, or various other substances, will produce a dense and waterproof material. Concrete rendered impervious by one of these methods, may be used in forming the larger sewers in place, and in making the manholes and other appurtenances. Also, in the construction of pump wells and disposal works; and in various instances, this method may be supplemented by the use of bituminous exterior coatings.

In Baltimore where the most elaborate and extensive system of sewers now under way in this country is being constructed, and no detail is being lost sight of, or expense spared, which could render the system more perfect, the intercepting sewers are being treated with a complete waterproof coating.

The waterproofing of sewers is perfectly practicable, and the problem is rather one of the best methods for a given case than it is of any other nature.

On the economic side, so far as direct saving may be effected

Mr. Hammond. by waterproofing, it is of interest to observe the extent to which ground water has been found to enter sewers not waterproofed, and a few instances are added in which determination was made of the extent of the leakage.

AMOUNT OF GROUND WATER INFILTRATING INTO SEWERS.

(Taken from reports of Engineers.)

New Orleans, La.—The plan provided for the removal of .003 cu. ft. per sec. per acre of ground water, or 1,250,000 gallons per sq. mile per day.

The annual report of the Sewerage and Water Board for 1909 states at page 54 that Sewerage Pumping Station "A" discharges in dry weather an average of 20,000,000 gallons per day, of which "probably 15,000,000 gallons is ground water or seepage, which is being absorbed continuously from the soil of the city, lowering its moisture line in the soil and vastly improving its sanitary condition."

The sewers of New Orleans have been constructed with great care, on the separate system, and the system is, in the writer's opinion, one of the finest in the world. It is so located that the pumping of ground water is a distinct advantage to the city. Therefore, no waterproofing was undertaken, but the joints were all made as tight, and the work was constructed as water-tight, as such work can be made without special provision for waterproofing. The surface of the city is lower than the surface of the Mississippi River, except along the levees. However, the result well illustrates the freedom with which ground water enters well constructed sewers that have not been specially waterproofed.

The pipe sewers, from the smallest size up to 36 in. in diameter, are of the very best quality of vitrified stoneware pipe. The main sewers are principally of brick although there are also several miles of concrete sewer of sizes from 3 ft. 9 in. to 5 ft. 9 in. in diameter.

Brooklyn, N. Y.—The greater part of this sewerage system is in such high and dry ground that there is but little trouble from ground water. The system consists of nearly 1,000 miles of sewers, of which nearly half is cement concrete pipe in sizes up to 24 in.

in diameter. There are also about 300 miles of vitrified pipe sewers. The maintenance cost per mile per year is about equal on each kind of pipe, and is very low. Maintenance cost of the brick sewers per mile is about four times as great. Mr. Hammond.

The portions of the system in the low laying portions of the city, which were nearly all built before the localities sewered became part of the city by annexation, admit considerable ground water. The 26th Ward, or East New York District, formerly the town of New Lots, before annexation, contains a population of about 200,000, and all of its sewage in dry weather is pumped and treated. The dry flow varies between 15,000,000 and 20,000,000 gallons per day, in accordance with the season, and with the level of ground water; from 5 to 20 per cent. of the flow is estimated to be ground water.

The system consists of brick main sewers, and of vitrified pipe and cement concrete pipe collecting sewers. The two kinds of pipe are used in about equal quantities.

The same conditions obtain at Coney Island, where the leakage into the sewers ranges from 20 to 50 per cent. of the flow, all of which has to be pumped. The Sewer Bureau is relaying much of this leaky system with sewers having carefully waterproofed joints, etc.

Reading, Pa.—The water supply is 127 gallons per capita. The sewage is 163 gallons per capita. The writer is informed that the ground water entering the sewers amounts to 600,000 gallons per day, or 5,172 gallons per mile per day.

The sewer system is constructed of terra cotta pipe, and brick and concrete mains. The system is practically all new.

Providence, R. I.—The report of the City Engineer for the year 1905, page 37, gives the amount of sewage pumped for the year as follows:

Daily average for the year	21,706,889 gallons
Daily average for wet weather	32,131,655 "

An increase of about one-half in wet weather. The system is a separate system.

The sewers, from 16-in. in diameter up to the largest size, are constructed of brick. The pipe sewers range in size from 6 up to

Mr. Hammond. 15 ins., and are partly iron, partly terra cotta, etc. There is about one mile of 18-in. pipe sewer. The report quoted states that there are 40.45 miles of brick sewers and 134.8 miles of pipe, to which should be added 28 miles in the "improved sewerage system," making about 203 miles.

Baltimore, Md.—In the new system being installed, notwithstanding careful waterproofing of intercepters, and the use of waterproofed joints, provision is being made in the size of the sewers for the entrance of some ground water. The Board of Consulting Engineers say, in their report to the Sewerage Commission (1906):

> From our experience with this subject in other places, it is our opinion that in the earlier years of the new works, the total amount of the sewage would not exceed the water supply. In subsequent years, when the sewers are extended into less thickly populated portions of the city, and buildings are more generally connected with the sewers, *the sewage increased by ground water* will be materially in excess of the water supply.

They advise providing for 150 gallons of flow per capita; and planning the sewers to flow half full with this quantity; or in other words, they provide for 300 gallons of flow per capita, with the sewers flowing full. The system is a strictly separate system, vitrified clay pipe is used for all pipe sewers.

Columbus, O.—Ground water entering existing sewers, when ground is wet after storms, is about 100 per cent. of the flow; during storms it ranges from 100 to 296 per cent. of the dry weather flow. System is built of brick, terra cotta and concrete.

Kalamazoo, Mich.—Ground water entering sewers is about 20 per cent. of their capacity. System is formed mainly of terra cotta pipe.

Norfolk, Va.—Ground water forms 60 per cent. of the pumping. The sewers are said to be of brick, terra cotta pipe and iron pipe.

Canton, O.—Main sewer admits ground water at the rate of 70,000 gallons per mile per day.

Brockton, Mass.—The ground water flow is said by reports to have been 400,000 gallons per day from 16 miles of sewers.

Plainfield, N. J.—The daily dry weather flow amounts to 1,400,000, which is increased to 1,700,000 gallons in wet weather.

The system consists of 43½ miles of sewers, constructed of terra cotta pipe and some iron pipe. It is a separate system. Mr. Hammond.

Red Bank, N. J.—Strictly separate system. No storm water is supposed to enter, but after a shower, the flow quickly doubles in quantity. Terra cotta pipe, and iron pipe are said to be the material used.

Vineland, N. J.—Strictly separate system of sewers constructed of terra cotta pipe. The flow in wet weather is nearly double that in dry.

Westfield, N. J.—Strictly separate system of sewers constructed of terra cotta pipe. There are about 15 miles of sewers 8-in. to 24-in. in size. The ground water entering the sewers increases the ordinary flow about 50 per cent.

This list might be extended indefinitely and probably would include every sewer system in the country installed previous to 1900 and many since installed. It would serve no useful purpose to add to it. I regret that I have not been able to include more data, relative to concrete pipe, but it does not seem that this material has given as much trouble as has terra cotta and brick work. The reason for this probably is that cement pipe has but seldom been used in the smaller sewers, of which the separate system is so largely composed. Where used for sewers, its record upon the whole is at least equal to the best clay product.

The old discussion as to which is the better, clay pipe or cement concrete pipe, is still with us, and we cannot say that it looks as though it would soon be settled.

MR. RUDOLPH HERING.* (By letter.)—The discussion of Mr. George T. Hammond on Waterproofing and Infiltration of Water in Sewers has been sent to me for discussion. Mr. Hering.

It is true that vitrified pipe, cement and brick sewers as a rule admit ground water when below the water table and permit leakage when above the same. In the latter case it is always desirable and sometimes imperative to make sewers watertight. That this can be done has been proven years ago by building sewers in a cradle of properly proportioned and well compacted Portland cement concrete.

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* Consulting Engineer, New York, N. Y.

Mr. Hering. As Mr. Hammond correctly says, gradual silting up of small openings and closing of the leaks frequently occurs, but there are many cases where it does not occur or the sewer does not remain permanently sealed.

Waterproofing can be applied in three ways. One way is to have the material of the structure and the joints impervious. This will prevent leakage in either direction. Another way is to coat the outside of the sewers if the object is to prevent infiltration of ground water into the sewer. The third way is to coat the inside of the sewer if a leakage of the sewage into the ground is to be prevented.

According to these three demands, different methods of prevention may be used. Mr. Hammond does not give the details of the methods of waterproofing which he says are "perfectly practicable." In view of the various materials recommended for this purpose and in view of the various recommendations made and exhibits at the recent expositions both in Europe and here, it seems proper to have the tried methods not only enumerated, but well explained for general information, and also to add whatever tests of waterproofing have been made under various conditions. The present information is that the degree of success has not as yet been quite satisfactory.

Mr. Hammond has very properly indicated that not only pipe sewers, but also brick sewers, leak and it would also be proper to say that concrete sewers, having a mixture of ingredients which allows the same porosity, also leak. It would also be well to have some experience data on the best mixtures of the ingredients to prevent such porosity, both under ordinary conditions of making concrete and under conditions where concrete pipes are molded with some pressure or special tamping.

It would also be interesting to know what effect upon the porosity of cement pipes is had by varying the proportions of water in mixing. Some curious results have now and then been reported, showing that sometimes a very wet mixture and sometimes a rather dry mixture have produced a greater porosity than a quantity of water in between.

Mr. Hammond seems rather hard on iron pipe when this is used for sewerage purposes, but he only states what has been the

experience of many engineers. The only excuse for using iron seems to be the necessity of bridging an opening and where there is a very small dimension allowable between the inside and outside of the sewer pipe. So far as durability is concerned, iron is as a rule inferior to stoneware. Mr. Hering.

Mr. Hammond gives a large number of cases where in different cities observations have been made on the amount of ground water leaking into sewers. This information is obtained from numerous reports, but only very general deductions can be drawn from it. So much depends upon the character of the ground, the amount of ground water contained in the same, and upon the way the sewers have been laid and the joints made, that it is very difficult and almost impossible to draw any further conclusions than that sewers may occasionally act as very efficient drains for the surrounding country. I could add to this data the case of East Orange, N. J., where before a single house connection was made to the sewerage system, the pipes ran about half full of clear ground water for the removal of which no provision was intended nor necessary.

Mr. Hammond uses the words "terra cotta" in designating most of the vitrified sewer pipe. This is not correct. It would be better to use a more characteristic and descriptive term. The word "terra cotta" was formerly, and more correctly, applied when slip glazing was used and the material was not vitrified.

MR. KENNETH ALLEN.* (By letter).—When it is realized that, as pointed out by Mr. Hammond, the dry weather flow in sewers is increased in many systems by from 50 to 100 per cent. by infiltration, the importance of watertight work is apparent, especially in those cases where the sewers are far below the ground water level and where the sewage is pumped or treated before its discharge. And it is well to remember here, that although sewers may be made almost impervious, unless house connections are laid with equal care, large volumes of ground water may enter through this source. For this reason it is desirable that the construction of house connections should at least be made under the supervision of the municipality. Mr. Allen.

The amount of infiltration is a function of the head of ground

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Mr. Allen. water on the sewer, of the linear measure of joints in the case of pipe sewers and of the superficial area of the interior of the sewer in the case of brick or concrete sewers. Unfortunately, there is little information by which an estimate may be made based on these fundamental considerations. The volume is usually stated as so much per acre, per square mile or per mile of sewer, per capita or percentage of dry weather flow.

In Providence the infiltration has been estimated at 47 gallons per capita daily.

In Waterbury the infiltration to a concrete sewer about 4 ft. 5 ins. square laid under a head of from 4 to 12 ft. amounted to one gallon per second for every 400,000 sq. ft. of interior surface, or 0.218 gal. per sq. ft. per day.

The infiltration to 140 miles of sewers in Massachusetts towns has been estimated at 40,000 gallons per mile daily. Taking the larger systems of the state it has been estimated at 70,000 gallons per mile.

In the Metropolitan system near Boston it has averaged 40,000 gallons per mile before making connections.

In their report on a system of sewerage for the city of Baltimore in 1897, Messrs. Rudolph Hering and Samuel L. Gray recommend an allowance of 750,000 gallons per square mile in the low-lying districts near the harbor and 100,000 gallons per square mile per day in the other parts of the city. In 1906 Mr. F. P. Stearns proposed an allowance of from 30,000 to 80,000 gallons per mile of sewer daily. In later estimates 12 gallons per capita have been assumed.

Mr. C. E. Grunsky estimated for the sewerage of San Francisco a daily volume of 55,300 gallons per square mile for pipe sewers and 166,000 gallons per square mile for brick sewers.

In Peoria, Ill., the infiltration was estimated at from 50,000 to 100,000 gallons per mile for sewers in wet ground. In the case of brick sewers it was estimated at 15 gallons per square foot.

An extreme case occurred in New York City "in which the 24-in. outlet to about 5 miles of 8 to 24-in. sewer in ground full of water was running full and the water standing several feet above the top of the sewer in the manholes, before any houses were connected, the fact that there was a 6-in. sub-drain evidently being

of little importance; giving a calculated approximate flow of more than 1,300,000 gallons per mile per day."* Mr. Allen.

Table I gives some instances in the United States based, generally, on the mileage of sewers.

From a consideration of the above figures, it is seen how serious a matter the ground water question may become under unfavorable circumstances; but if we assume the sewer to be laid with care and

Table I.—Infiltration to Sewers.

Town.	Gallons per mile daily.	Remarks.
Alliance, O.	195,000	15 in. diameter.
Altoona, Pa.	40,814 to 86,592	30 in. diameter.
Brockton, Mass.	20,860	= 27 per capita. Brick sewer in wet ground admitted 4,000 gallons per mile daily before connections were made; 117,000 gallons in wet weather.
Canton, O.	26,500	11 miles of sewer.
Clinton, Mass.	32,500	= 62.4 per capita.
Concord, Mass.	43,000	18 per cent. of sewers from 0 to 16 ft. below ground water.
East Orange, N. J.	8,640	Vitrified pipe. Before connecting.
	573,000	Brick. Before connecting.
Framingham, Mass.	31,180	= 65 per capita.
Gardner, Mass.	45,875	= 156 per capita.
Madison, Wis.	48,000	Trunk sewer in dry weather.
Malden, Mass.	50,000	= 65 or 70 per capita. 60 per cent. in wet ground.
Marlboro, Mass.	59,540	= 144 per capita.
Natick, Mass.	77,420 to 100,000	= 209 per capita.
North Brookfield, Mass.	17,000	12 in. pipe.
New Orleans, La.	31,800	This amount was about doubled during certain periods.
Peoria, Ill.	100,000	Vitrified pipe.
Rogers Park, Ill.	1,240	6 in. pipe. Head of from 1½ to 9½ ft.
Schenectady, N. Y.	5,000 to 20,000	4,000 per mile before connections were made.
Washington, D. C.	.....	.0015 cu. ft. per second per acre after a very wet season.
Westboro, Mass.	1,320,000	15 in. pipe in wet ground.
Worcester, Mass.	31,700	
	800,000	14 gallons per day per sq. ft. of interior brickwork (mentioned by A. P. Folwell, *Eng. Record*, Vol. 48, p. 564).

under average conditions it would appear that an allowance of from 25,000 to 50,000 gallons of infiltration per mile should be allowed for above the maximum flow of sewage used in designing new sewers, depending on the permeability of the soil, the static head, the diameter of the sewer, its material and the method of construction. Granting that under certain conditions, such as a wet soil and where the sewage is to be pumped or purified, it is desirable to

* A. P. Folwell, Eng. Rec., Vol. 48, p. 564.

Mr. Allen. prevent the entrance of ground water to the sewers, it follows that these should be as nearly impervious as possible.

In the case of vitrified pipe sewers, if these are sound and of good quality it only remains to secure an impervious joint. This is not practicable with natural cement. With Portland cement mortar following a gasket saturated with grout and carefully calked there will be very little leakage. The difficulty lies in ensuring perfect work, especially in a wet trench, and the possibility of cracking the joint by settlement in a soft soil. To overcome these objections various substitutes for the cement mortar joint have been used with more or less success.

With a firm bed or foundation a mixture of 1 part of fine sand with 1 part of flour of sulphur heated to about 230° Fahr. poured in the joint over a well calked gasket is said* to give a watertight joint, costing for an 8-in. pipe 12 cts., for a 15-in. pipe 19 cts., and a 30-in. pipe 30 cts. Possibly "leadite," which is somewhat similar in character and which the writer has used successfully for cast iron pipe joints, would answer equally well. Either of these materials will provide a strong but unyielding joint, so that in case of settlement an undue strain is brought to bear on the bell.

For yielding soils the writer believes some form of bitumen may eventually be found that will be both elastic and impervious. For a number of years Mr. J. H. Decker has used a paste made by kneading cement and pine tar pitch by hand until of the consistency of dough in the sewers of Atlantic City with success, the material being pressed into the joint by hand. The writer employed this method in laying about ¼ mile of conduit of 8 to 20-in. vitrified pipe conveying water from a series of artesian wells to a reservoir. Testing under a head of 3 or 4 ft., after completion the line was found to be almost absolutely watertight This joint, although setting quite hard, remains sufficiently pliable to conform to ordinary settlement without injury.

In Baltimore Mr. E. B. Whitman experimented with a joint which proved so satisfactory that a considerable amount of vitrified pipe has been laid in wet trenches on the sewerage of that city with bitumen joints. The specifications provide that the inside of the bell and at least 4 in. of the spigot end shall first be painted with a

* Eng. Rec., Vol. 49, p. 336.

thin paint consisting of a solution of bitumen in gasolene or turpentine. Two or three pipe lengths are usually joined together before lowering to the trench by first centering and packing with a thin gasket of hemp or jute well caulked and then filling the joint full of hot bitumen, heated sufficiently to expel bubbles but not so as to render it brittle. The bitumen may be the refined natural or artificial material or a mixture of both, but must not have a melting point lower than 170° Fahr. or be brittle when cooled to 30° Fahr. It must also be of such a consistency that a fragment will not materially alter in shape when kept in ordinary temperatures and must have a moderate degree of resilience. Mr. Allen.

As a test it is specified that 3 lengths of 8-in. pipe joined in this way and resting on supports 7 ft. apart shall carry a center load of 200 lbs. without failure or excessive deflection, and that after such test the joints shall show no leakage when subjected to a hydrostatic pressure of 5 lbs. per sq. in.

Asphaltic joints have been used to a considerable extent in England and Germany, and the writer has long believed that a field would open for them in this country under conditions where cement work would be injured by washing or, later, by cracking.

With larger sewers it is desirable to employ concrete in preference to brick if watertight work is desired. There is no question but that reasonable tight work can be secured by using correct proportions and a wet mixture, but additional safety may be had by the use of waterproofing compounds, by the judicious admixture of lime or clay to the concrete or by the application of a layer of waterproofing material. The writer believes that the latter safeguards are rarely required in sewer work when good work can be relied on by contractor and inspector.

In the construction of the west low level interceptor at Baltimore, Md., waterproofing was applied to the interior surface of the concrete and a brick ring added inside to protect this. The diameters on Section 1 and 2, which lay along the waterfront with the invert from about 11 to 13 ft. below tide level, were from 74 to 84 in. The invert was first swabbed with hot Trinidad asphalt upon which was placed a layer of 10 oz. burlap. This was then rubbed down and painted with hot asphalt, a second layer of burlap applied and this was then brushed down and saturated with a third

Mr. Allen. coat of the asphalt. The brick ring was then laid complete and the waterproofing carried over the brick arch. The concrete arch was finally laid on this.

Some difficulty was experienced at first from the water which dripped on the tops of the side walls and found its way between the waterproofing and concrete of the invert, forming blisters 1 or 2 in. in diameter. Where this occurred the blisters were pierced, emptied and patched. To prevent its recurrence channels were left in the tops of the side walls to cut off the flow from outside and the lap of the burlap rolled up and covered with canvas to keep it as dry as possible.

In forming the joint with the waterproofing of the arch the layers were lapped in pairs and an additional strip of burlap added outside, making the waterproofing at this most troublesome location 5-ply.

To drain the channels left in the side walls 3-in. terra cotta drains were inserted, occasionally running down through the side wall to a ¼-bend and thence to the gravel near the underdrain.

If the burlap got wet, as was sometimes inevitable, it was dried with a gasolene torch, although in extreme cases this was not practicable. To dry the invert before waterproofing, if too damp, gasolene was sprinkled on and lighted, but in sparing quantity to avoid injury to the concrete. At other times it was merely sprinkled with dry cement, which absorbed the moisture and was then brushed off.

Estimates of the actual cost were made from time to time with the following results:

Cost of waterproofing	84 in.	sewer.....	$1.60	per lin. ft.
" "	80 "	"	$1.53	" "
" "	76 "	"	$1.45	" "
" "	74 "	"	$1.41	" "

Cost per square yard:

Burlap @ 6.9 cts. per sq. yd.	17 cts.
Asphalt @ 25 cts. per gallon	48 "
Labor	18 "
Total	83 cts.

The force employed usually consisted of a foreman, 2 laborers and 2 helpers.

On Section 3 the cost was reduced by the substitution of tar pitch for asphalt. Difficulty was experienced in securing a material that would be neither too soft at high temperature nor brittle at low temperature, but in other respects the results appeared satisfactory. The tar pitch cost $8 per 1000 lbs., resulting in the cost for this material of but 15 cts. per sq. yd. of waterproofing. Mr. Allen.

After the completion of the 3590 lin. ft. of interceptor, about 100 ft. of which passed under the bed of a tidal stream, the leakage, including that from over 2700 ft. of house connections and a considerable inflow around a bulkhead at the upper end of Sec. 2, amounted to 11.16 gallons per minute, or 0.223 gallons per sq. ft. of interior surface of interceptor (only) per day. The actual infiltration through the interceptor was practically nothing, as the water observed entered chiefly around the bulkhead mentioned, through the laterals or at the junctions with the interceptor. Probably 1/10 gal. of ground water per sq. ft. per day would be a liberal estimate in estimating on similar work. Considering the relative elevation of the ground water and the great difficulty in keeping the burlap at the spring-line dry while turning the brick arch, the final result was most satisfactory.

Mr. J. R. Riedel.* (By letter.)—The discussion by Mr. George T. Hammond upon the subject of the infiltration of water into sewers is both highly interesting and quite timely. Perhaps, there is no more serious problem confronting the sanitary engineer when engaged upon the design for systems of sewerage which will lie below the water table of the locality than the preparation of proper specifications, and their subsequent enforcement, for the laying of the sewers in such manner as to exclude ground water. Mr. Riedel.

Many attempts have been made to lay sewers so that they would be watertight, but up to quite recently, so far as one may learn from published reports, no sewer had been built below the water table, for any considerable length, where there was any positive assurance that the sewer was watertight. As has so often been pointed out, one of the most serious causes of leakage has been the faulty joints between the sewer pipes as well as upon the

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Mr. Riedel. house connection drains. It would appear as though no practical difficulty should present itself in the work of making our sewers proof against leakage, with the several methods of waterproofing now available, if the expense of doing so is justified, and this must be considered in each case. It would appear as though the principal problem to be met, so far as making pipe sewers waterproof, was in the mechanical process of making the joints. Here, however, seems to be the crux of the whole matter. Every engineer supervising such work and every contractor executing the work, knows how difficult it is to force the ordinary sewer pipe layer to give to the joint the attention which it should have and to allow it proper time to set before being disturbed by covering. Mr. Hammond cites many cases of leakage found to exist in the sewers throughout the country, and it might be of interest to here mention some observations made by the writer while engaged for the Brooklyn Bureau of Sewers in an investigation which was made in 1906 concerning the tightness of the joints of the pipes used in some of its sewers. The sewers of the old City of Brooklyn are on the combined system, and it may be said that generally none of the pipe sewers lie below the ground water level. In the newer wards and districts, the low lying country and the considerable distance to tidal water into which untreated sewage may be discharged, and the requirement that the sewage in many places must be pumped, has led to a design for a separate system of sewers. The sanitary sewers will to a considerable extent lie below the water table, and it is therefore of great importance economically to exclude all ground water.

Up to 1906 the Bureau of Sewers of Brooklyn specified that the joints of all pipe sewers, both vitrified pipe and Portland cement concrete pipe, should be filled with a mortar composed of equal parts of cement and sand and that a collar of this mortar should be left about the outside of the joint. The sewers were laid under the usual supervision and inspection and it was desired to know whether any leakage took place from the sewers. All of the sewers examined lay above the water table.

In all, some thirty lengths of sewer, ranging in size from 12 to 24 in. were examined. Each length or section lay between two manholes which were from 100 to 130 ft. apart. Bulkheads con-

sisting of 2-in. planking covered with a rubber packing cut to fit the interior of the pipe were placed in the manholes at each end of the sections so as to include both manholes in the test. The interior of each manhole was plastered with a coating of Portland cement mortar of one to one proportions and allowed to set for at least one day. Water from the city water supply through a 4 in. hose was let into the section and when the water rose to a height of about 3 ft. in the up grade manhole, it was shut off. Simultaneous readings were taken in each manhole with hook gauges as often as the rapidity of the leakage warranted. Mr. Riedel.

Usually readings were taken every minute and continued for an hour unless the leakage was so rapid as to reduce the level below the top of the pipe in the up grade manhole before that time. The sections of sewer examined were in different parts of the borough and the soils in which they lay were sand, sand and gravel, loamy sand, and clay.

The results were plotted and were very carefully studied, and the general conclusion reached was that the only sewers examined which did not leak were those which were surrounded by clay. Those which were laid in sand were found to leak by an amount in some cases equal to 60 per cent. of the carrying capacities of the sewers. In three cases it was found impossible to fill the manholes to the height desired; the water having been permitted to run in for more than an hour. In no case where the sewer was laid in sandy soil or in sand and gravel, did the leakage fall below 15 per cent. of the carrying capacity of the pipe. That all the leakage did not occur through the joints of the pipe is very likely. Some may have issued from the spurs left for future house connections, but the specifications required that all such spurs should be stoppered with covers cemented into place. The sewers examined were of both vitrified pipe and cement concrete pipe and there was little to choose from, so far as the material of the pipe was concerned.

While the sewers tested as above would probably leak but little under the conditions of ordinary service, there is always a possibility that extra heavy storms may put the sewers of the combined system under pressure and duplicate the effects observed. But little leakage out of these sewers occurs under ordinary service, as they seldom run more than one-third full, and it was observed that

Mr. Riedel. at this depth there was but slight leakage in any case. Where ground water is present, and rises so that the water table gives a head over the sewers on the outside of its walls, comparable to that which in these experiments was produced artificially inside of its walls, we should expect as much leakage into the sewer as in these experiments took place outside, and to determine this was the most interesting point of the experiments.

The observations made in 1906 led to a modification of the specifications relating to the jointing of pipes and it is believed that the joint as made at present by a hemp or oakum gasket saturated in neat Portland cement grout and rammed into place by means of a special tool, and then completed with a rich mortar (1 to 1½) will produce a sewer that will not leak at the joints under ordinary pressures to be expected. For the sewers on the separate system which will be below the ground water level, it is proposed to use a joint consisting of a bituminous, or an asphaltic, or a vulcanized substance. These substances have given satisfaction in many places, especially abroad, where they have been used for a number of years. The life of these compounds is not known, but it is believed that it will be long enough to make their use profitable in any case, especially where a large volume of ground water is present under considerable head.

Mr. Pitkethly. MR. DAVID T. PITKETHLY.* (By letter.)—While the leakage of ground water into sewerage systems, as well as the escape of sewage into the surrounding soil has always gone on since sewerage and drainage has been in existence, the importance of having the sewers made watertight is just dawning upon us. The admission of ground water into a sewer is advisable when the water table lies so close to the surface that for sanitary reasons the lowering of the ground water is necessary and no other means of disposing of it can be devised.

In the present era of strictly separate sewerage system and sewage disposal, the matter of ground water is a serious one, as usually the sanitary flow has to be pumped and every gallon of water handled means additional cost for pumping as well as necessitating increased capacity in tanks and filtered beds at the disposal

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works. The writer during his connection with the Sewer Department of Boston, Mass., and Sewer Bureau of Brooklyn, N. Y., has always been an advocate of waterproofing all structures to be placed below the water table. Mr. Pitkethly.

For many years the writer has been familiar with sewer manholes laid with brick bottoms, walls with no waterproofing coat on the outside, and a sand joint around the pipe where it enters the manhole, all sources of leakage.

The Brooklyn Sewer Bureau has now adopted a standard manhole for pipe sewers with a concrete bottom, and walls at least 12 in. thick up to a point above the water table, or of 12 in. of brickwork thoroughly coated on the outside with ½ in. of Portland cement mortar, making them practically watertight walls and bottom. The sand joints are done away with and a tight cemented joint is now made as the provision of sand joints is not necessary where good foundations are to be had, or a cradle is provided under the sewers.

One of the most serious means of admitting water into a sewer system is through house connections and drains. The sewers when laid are usually provided with a spur, opposite every point where a connection is expected which is stoppered and buried with the sewer.

A record is kept of the location of each spur, and when the house is to be connected, the work is usually done by a plumber, who gets the spur location from the Sewer Department, and proceeds to dig for it. His chance of finding the spur when he digs is sometimes well taken, and the spur is found. Then the danger is only in leaky joints in the house drain which are seldom properly calked. But if the spur is not found, as is very often the case, the plumber usually breaks into the sewer, inserting the end of the drain, thus choking off the sewer and leaving a jagged, irregular opening in the wall of the pipe sewer which no amount of care can make tight.

The leakage through the walls of pipe sewers is negligible, but the joints are always a source of trouble. This has been met in Brooklyn as far as possible by extra wide and deep sockets on the pipes, thus allowing for the proper insertion of an oakum packing soaked in neat Portland cement mortar, and firmly rammed into

Mr. Pitkethly. place. The remaining portion of the annular space is then filled at bottom, sides and top with one to one and one-half Portland cement mortar, forced into place with a calking tool and the joint then wiped smooth to a bevel. The utmost precautions are taken and all work is done under most rigid inspection.

In the matter of brick sewers, about all that can be done is to give them a coating of Portland cement mortar, which should make them reasonably tight, providing the remaining portion of the work is properly done.

For concrete sewers it has been the practice in Brooklyn to use a 1:2:4 mixture laid very wet which has given good results as far as our observations have shown.

The result of these measures has usually been satisfactory under average conditions of sewer construction, but there are special cases requiring special treatment. Where sewers and other structures are wholly below ground water each case must be provided for on the merits of each individually. Experiments have been made and are still under way for the purpose of securing improved methods of making tight pipe joints and in waterproofing exposed surfaces.

It may be remarked in conclusion that well made concrete correctly proportioned for density tends to become more and more impervious to water as time passes, while almost every kind of surface applications formed of bituminous materials probably tends to deteriorate.

Mr. Kaufman. MR. GUSTAV KAUFMAN.* (By letter.)—Mr. Hammond's excellent discussion is not only instructive, but is replete with many suggestions which should awaken profitable discussion among sewer engineers. One of the most salient features, to the writer's mind, is that the cities referred to generally use vitrified clay pipe exclusively and that, nevertheless, there is a large infiltration of ground water into the sewer system. Inasmuch as vitrified clay pipe is not porous, the data furnished by Mr. Hammond confirms the opinion of the writer that there is no infiltration into sewers through the walls of the pipe, but occurs on account of broken pipe, open joints and manholes.

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Mr. Kaufman.

The writer is the engineer for the company which has made most of the concrete pipe used by the City of Brooklyn, probably four hundred miles, varying from 6 to 24 in. in diameter. The pipe that we have furnished for Brooklyn is made by a machine tamper. The mixture is one part cement, one part sand and three parts trap rock screenings, in which the dust is about twenty per cent. In order to permit of machine tamping the mixture is made quite dry, with the consequence that the pipe, when subjected to hydraulic pressure, shows, at first, considerable porosity. At 5 lbs. per sq. in. water begins to ooze quite freely through the walls of the pipe; but if the pressure is maintained the pipe soon becomes tight and we have frequently had pipe perfectly tight under an hydraulic pressure of 25 lbs. per sq. in. the limit of our testing apparatus for this purpose. The walls of the pipe, which were tested, were about one inch thick.

We have already embedded pipe in sand, so as to duplicate conditions as exist in the ground, and saturated the sand with running water under a pressure of about 5 lbs. per sq. in. This would represent conditions where moving ground water exists. Absolutely no water entered the pipe through the walls, although under test for many days.

I am, therefore, of the very strong opinion that no ground water, under the conditions that are found in the trench, will ever enter concrete pipe sewers through the walls. All such water must enter through broken pipe, open joints or at manholes. This opinion is verified by experience in irrigation and drainage projects.

Holding this view, I am not an advocate of expending public funds for applying any waterproofing material, either to the exterior of the pipe or in the mixture of the concrete. It may be of interest to add that we have made 12-in. pipe, with 1-in. walls, putting in the mixture a percentage of silicate of soda. Under test such pipe showed no leakage whatever up to an hydraulic pressure of 80 lbs. per sq. in. The pipe, however, ruptured under this pressure.

I am convinced that the leakage through the walls of sewer pipe, even under a suction by pumping of 10 or 12 lbs. per sq. in., would be practically nothing, and that, by the making of tight joints and probably waterproofing the brick manholes, a very efficient separate sewer system can be obtained.

REPORT OF THE COMMITTEE ON ROADWAYS, SIDEWALKS AND FLOORS.

Last year the Committee presented certain specifications which have not been thoroughly tried out, owing to the short time intervening between this meeting and the last one, therefore it is considered to be out of order to recommend revisions in these specifications at this time.

It seemed desirable that the Committee present a specification on the treatment of concrete floors with the idea of eliminating the dust nuisance. Sufficient reliable data was not available, however, to justify recommendations. Without some very serious thought and possibly some extensive experiments and investigations, it is doubted whether recommendations can be made even a year from now that will be sufficiently definite to justify adoption.

Respectfully submitted on behalf of the Committee,

C. W. Boynton,
Chairman.

DUSTLESS CONCRETE FLOORS.

By L. C. Wason.*

The subject of dustless concrete floors can be divided into three divisions. 1. Causes for dusty floors. 2. Elimination of dusty floors during construction. 3. Remedy for dusty floors.

1. Causes for Dusty Floors.

A mortar surface which is smooth and permits dust to collect on it from outside sources is oftentimes unjustly blamed for dustiness, whereas a wooden floor allows part of the dirt collecting upon it to drift into the cracks and leave the rest of the floor apparently much cleaner than it really is.

What is a dustless floor? It is not one that is absolutely free from dust. It is one which wears out very slowly, because every substance will wear more or less in course of time. It is a floor which wears so very slowly that the amount of dust which accumulates is imperceptible and is not troublesome or negligible from a practical point of view. The majority of granolithic sidewalks are dustless pavements which are worn by the feet of people with very little sliding or abrasion by the rolling of wheels. Usually around mercantile buildings the trucks have small iron wheels, seldom more than 6 or 8 in. in diameter, with a face of 2 or 3 in. and sharp on the edge of the tread. The hardest service is dragging substances of one kind or another over the surface, yet they fulfil the above requirements for a dustless floor.

A material must be used which will resist these three types of wear. It is pretty well known that Portland cement itself will not withstand wear as well as natural stone will. We frequently see the glaze which is troweled up in finishing a floor quickly worn away, exposing the aggregate of which the concrete or mortar is composed. If the wearing of this surface finish is objectionable

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when the building is put into service it can be removed by grinding or polishing, the way terrazzo floors are polished with a rubbing stone, because it is pretty sure to come off so quickly as to form a dust.

Dust is sometimes caused by the fine particles of which the mortar is composed. If fine sand or fine stone dust is used these particles which have sharp points are ground by wheels of trucks so as to produce dust, or else by being forced into the surface below if it be a little softer than the particles themselves, some of the adjoining particles are separated and disintegrated, thus producing dust. Therefore, one way to avoid a dusty floor is to avoid very fine particles.

The smallest particle of stone which will stand ordinary wear, in the writer's opinion, is one about $\frac{1}{8}$ in. in diameter. This, if a hard stone, is sufficient to carry the weight of a four-wheel truck carrying load of 10,000 lbs. Such a wheel with a 2-in. face would have a bearing of about 2 in. by 1/32 in. or about $\frac{1}{4}$ sq. in., which is about four times that of the stone in question. So that if the wheel bore uniformly across its area there would be very little chance for abrasion. If it bore on a point it probably would abrade.

Thus, to get a surface just as nearly free from dust as possible we want one which is as free from abrasion as possible. Proportion the aggregate so that as large a portion of the surface as possible is stone. It is possible with care to get a surface of 90 per cent. stone and 10 per cent. cement. The cement, which wears a little faster than the stone, wears to a trifle lower level, so that the stones really get the major part of the wear. But such a stone may become somewhat slippery and this is a danger which, especially on sloping sidewalks, must be guarded against. In inside building construction where the floors are dry, it is not an objection but rather an advantage. So that to obtain a dustless floor there may be some conditions which are not of universal value, namely, being slippery.

There are several conditions which render a floor dusty which are independent of the aggregate. They are largely matters of manipulation. If the mortar is mixed too dry the surface is porous, the aggregates are not well bonded and will work loose pro-

ducing a dusty floor. On the other hand, if mixed too wet, laitance is brought to the surface, which is soft and spongy, readily wearing. Troweling may be delayed so late that the cement is damaged in quality and therefore renders the surface softer. Again, if the surface is exposed to very rapid drying it is apt to be impaired in quality, which in turn will render it somewhat dusty.

The use of dryers, sprinkling the surface with neat cement or a combination of cement and fine sand (in the proportion usually of 1:1), over the surface to absorb excess moisture, renders the surface dusty, because these are the particles which readily wear away and which do not bind well with the surface below, and this, therefore, should be strictly avoided.

Working the surface after it has been spread too long a time, when the cement has reached partial or final set, impairs its quality so as to render it dusty.

If a granolithic pavement is spread as a second operation, in two-coat work, over a first coat which has set hard and is nearly impervious to moisture, one can practically guarantee that the filler will be dusty under all conditions, at all times. The writer believes it is practically impossible to build such a floor and make it free from dust; because as the mixture cannot soak through the base it has to come to the surface and thus renders the surface spongy and soft. The nearest approximation in the writer's experience to render such a floor dustless has been to mix it almost as dry as a building block is mixed, then tamp it thoroughly, because only with extreme tamping is enough moisture brought to the surface to trowel. The excess of this moisture will soak back into lower portions of the coat and allow the surface to be troweled without producing a soft, spongy top.

The subject of thickness of top coat of floors may well be considered here. Specifications sometimes require the finish to be as thick as 1½ in. and very frequently as thick as 1 in. The writer believes that ¾ in. is sufficient. About 14 years ago a pavement was very well laid in front of a railroad station where thousands of people and also a great many teams passed every day. This was a sidewalk across the entrance to the carriage concourse of the station. Originally it was grooved, the grooves being about ⅜ in. deep, to prevent slipping of horses. After 14 years those grooves

are still visible. When a pavement will wear as well as that it seems to be a great waste of material to make it more than ¾ in. thick. Making a 4-in. pavement with a 3¼ in. base and ¾ in. of finish is a thoroughly first-class job, and saves enough cement over a 1-in. top to about pay for the safeguards which have just been described.

2. ELIMINATION OF DUSTY FLOORS DURING CONSTRUCTION.

Having considered these various causes which produce dust in floors, how can they be avoided in actual construction? With regard to materials any first-class Portland cement can be used. There is some belief that certain brands are better for granolithic surfaces than others; but the writer's experience has been that any brand of first-class cement that will pass the Standard Specifications for Cement is suitable. Eliminate the use of sand altogether, because bank sand generally contains dirt. If it contains organic matter it is sure to be fatal. The writer was called upon some time ago to examine and report upon a building which collapsed. On first examination, by rubbing the sand between the hands and throwing it into water, it appeared to be clean, but by careful laboratory testing it was found to contain 4 per cent. of vegetable matter. This was sufficient to cause the absolute failure of the concrete. If the particles of sand have dust upon their surface, which is hard to work off, the cement bonds to the dust and not the particles of sand which therefore are not thoroughly bonded and work out.

The stone is the base of the dustless floor and great care should be taken in its selection. A stone which is suitable for macadam roadways, is suitable for pavements. Therefore, in any city visit the highway department or the engineer and find out the stone of that particular locality which best resists abrasion. It is the one which is best for a dustless floor. Laboratory tests are made of these by the rattler process, which consists of an iron cylinder supported in such a way that when it is revolved the stone is thrown from one end to the other and rattled about. The amount of wear is measured, and that stone which withstands the wear best is most suitable for a dustless floor. These tests are carried on by various laboratories of the state and nation and are available for inspection.

Regarding size, take all which passes a screen with a ½-in. round mesh and is rejected on a 20-mesh screen, all finer than this being removed, especially the flour of the stone dust. Gravel screenings of a similar size may be used. Sand which is finer than these particles is rejected. Be sure the particles of stone or gravel are hard and durable. As they are mostly round when the pavement is put in service the wear comes on little round points, and it is a considerable period of time before they wear down to a flat surface, which will withstand wear as well as the flatter fractures of broken stone.

The water should be perfectly clean, free from acids, oils, etc. The mixture should be 1 : 1½ or 1 : 2 and such an amount of water as to make a rather stiff or plastic mortar. Then with fairly first-class workmanship in the mixing of materials, spreading and troweling, being careful not to carry the troweling too late in the set, it is safe to guarantee the floor to be free from dusting.

Regarding the cost of such a floor, stone free from dust costs from 5 to 10 cts. per ton more than the run of the crusher. I tried to induce two quarrymen to put in blowers to blow out the dust, but they found that it was inexpedient and expensive. By putting a fine jacket around the regular dust screens on the rotary screens, this jacket being used only when the stone was perfectly dry, it was easy to obtain stone which was free from flour at an expense of 5 to 10 cts. per ton more than otherwise. As a 1 : 2 mixture can be used instead of a 1 : 1, which is very common with other materials, the saving in cement offsets the extra cost of the stone, while it costs a little more for labor. On a general average the labor cost in finishing an ordinary floor is about 1 ct. per sq. ft., and because this top coat is of such a rough, coarse material, it adds about a quarter of a cent per sq. ft. to the cost.

Now this material is really a concrete, not a mortar, using mortar in the sense of cement mixed with common sand, that is, fine aggregates, this is cement with coarse material or concrete.

3. Remedy for Dusty Floors.

If after a floor has been laid it has become dusty how can it be remedied? Of course it can be taken up and done over, which

is very expensive and unnecessary. If the surface is given a coating or two of boiled linseed oil it will almost invariably be dustless. There are cases of floors being so dusty that if one had the patience, with a stiff corn broom (not bristle) it would seem almost possible to sweep the whole floor away, and yet they were absolutely cured from dusting for an indefinite period by the use of boiled oil.

Another method which the writer has observed used by a member of this Association who recommends it, is silicate of soda. Take a solution which would gauge 40 deg. Beaumé, dilute it with 4 or 6 parts of water and brush it on and allow to soak into the floor. After it has soaked in from 4 to not more than 24 hours, wash off all excess and give it a second application. Sometimes a third is necessary. A floor treated this way which has previously dusted has been cured and cured permanently. Silicate of soda is not as expensive as boiled linseed oil.

DISCUSSION.

MR. W. W. SCHOULER.—Mr. Wason spoke of using concrete for a ¾-in. top coat so dry as to require tamping, and it occurs to me that there is enough water on the surface of a concrete pavement to wet 2 or 3 ins. of dry mortar on top. How is the excess water disposed of without the use of a drier? **Mr. Schouler.**

MR. L. C. WASON.—In the particular case referred to, using a top so dry it had to be tamped, a finish was put on as a second coat over concrete which had set hard. When it is put on before the base has set, the water drips out below. If the concrete is on the ground the water sinks into the ground. If it is on wooden forms the excess water drips through the forms, and not much of it comes to the surface. I mix the finish somewhat dryer than many people do, plastic rather than a sloppy wet. **Mr. Wason.**

MR. C. F. MACEVOY.—I would like to ask Mr. Wason, in this mixture of stone screenings for a top dressing, with no stones larger than ½ in. or smaller than ⅛ in., what fills up the voids between the stones? **Mr. MacEvoy.**

MR. WASON.—The material, such as is usually obtained from a stone crusher, running from ½ in. down to ⅛, is so well graded that the cement itself fills the voids. The voids are less than ½ the total volume of the stone, perhaps 40 per cent., while the cement is 50 per cent. of the volume of a 2 : 1 mixture. Thus the cement itself fills the voids completely. The 10 per cent. mentioned referred to the exposed surface. For instance, terrazzo floors have been polished so that there is probably not more than 5 per cent. of surface area showing cement. Now in the granolithic, after the glaze left by the trowel on the cement has worn away, exposing the stone, you will find 90 per cent. of the surface is stone, although the original mix was 2 : 1. **Mr. Wason.**

MR. MACEVOY.—While it is preferable to place on the cement finish at the same time the slab is cast, it sometimes cannot be done. For instance, the cement finish is handled many times under a separate contract and the contractor has to wait until all **Mr. MacEvoy.**

Mr. MacEvoy. the brick and other heavy work is done before the cement finish is allowed to be placed, or the architect wants to avoid the dropping of brick and other materials on this cement finish and scratching it. So then, the top coat is laid when the building is entirely finished and then these floors are hard and the forms have already been removed. In that event the top of the concrete slab is first grouted and the cement finish laid wet, which when trowelled brings water to the top. How is that water to be removed?

Mr. Wason. MR. WASON.—The best way is to mix the finish so dry it has to be tamped, or no free water would be obtained. I would recommend where it is necessary to place the top coat long after the base has set, in order to avoid dust the use of a thicker top, preferably 2 ins., and then it is possible to use two mixtures. Use a rather coarse material mixed fairly dry, 1¼ ins. thick, and then spread a ¾-in. finish on this. The excess moisture will soak down into the first coat and will give a chance to finish the surface as it ought to be. Moreover, this top will be strong and durable even if it does not bond to the hard base. If a thin top ½ or ¾ or even 1 in. thick is put on, as is sometimes done, it will almost invariably be dusty, will not bond well with the base and will curl up at the corners if the surface is cut into blocks.

The bond can be made considerably stronger by sand blasting the base so as to cut the cement off, because cement will bond to clean aggregates and will not bond with neat cement. Clean off all dust and follow it up with grout and a fairly dry mixture will then bond well.

Mr. MacEvoy. MR. MACEVOY.—I would like to ask what it is in this water coming to the top of the cement finish and lying there that causes the wearing surface to dust off? How do you account for the absence of trouble from dusting on sidewalk construction? Is it because of faster evaporation, the sun evaporating the water instead of it lying there?

I understand the common linseed oil was recommended, not the boiled oil.

Mr. Wason. MR. WASON.—Common linseed oil. When it is boiled some thickening material is put into it so that it dries much quicker than raw oil and forms on the surface or in the top ⅛ or 1/16 in. of the concrete. I do not know what the chemical reaction is and

suppose the oil is somewhat saponified by the alkalies, but it gives a greasy gloss to the surface which stands wear until the cement can take care of itself. It probably causes the dusty surface to wear away much slower, so the dust is not noticed. I know of an extremely dusty floor that was so treated and after 12 years it is in excellent condition. Mr. Wason.

In regard to absence of dusting trouble in sidewalk construction it might be said that the dust is not so noticeable in the open air.

The water rising to the surface brings up the fine particles of cement and a soft, spongy substance from the cement called laitance. This exists in some cements more than in others and the water also works up the very fine particles of the aggregate. The water is removed by evaporation and this usually delays the time for finishing so long that the final setting of the cement is disturbed and its strength in that way impaired. All these various features severally or together make the surface dusty.

Mr. Edward D. Boyer.—I would like to bring out Mr. Wason's opinion as to whether this linseed oil application is better than the water glass application. Will not a walk treated with linseed oil after a certain time, while it might settle the dust, make the walk very slippery and almost impossible to walk upon? Mr. Boyer.

Mr. Wason.—I have had no experience with water glass and mentioned it only. I have seen it used and the result impressed me as being satisfactory. As an alternative I have had such good results with the use of linseed oil that I have had no inducement to use anything else. It does not make a floor slippery so that it would be objectionable in that way. Mr. Wason.

Mr. T. M. Vinton.—It seems Mr. Wason's paper could be summed up to mean that the only way a floor finish should be put down in a reinforced concrete structure, if applied after completion of the structural work, is to make it of sufficient thickness so the foundation would be strong enough to carry its own load; and if applied at the same time as the supporting concrete, that only sufficient top is needed to allow working to a finish. That has been our experience. Mr. Vinton.

It is up to the cement man, it is up to the concrete man, to simply serve notice on the architect who specifies the finish must

Mr. Vinton. be put down afterwards and will only allow 1 in. in thickness, and take the stand that he will not get the result desired.

Mr. Ketchin. MR. A. J. KETCHIN.—I want to ask Mr. Wason how he spreads this coarse aggregate in using it on the top dressing. If a straightedge is used would not the coarse aggregates roll and follow up after the straightedge?

Mr. Wason. MR. WASON.—A straightedge is employed in the usual way, only it requires a good deal more work because the concrete is rather stiff. It is done just the same as you do a soft mortar with fine aggregate, which works very readily under a straightedge and trowel. The following up of the coarse aggregates is one thing which makes it a little difficult to use this method, but the results I believe justify its use.

Mr. Boyd. MR. R. W. BOYD.—It seems to me we are after a dustless concrete floor, but were suggesting various expedients to get away from this trouble. We must start at the beginning in order to get the floor right. In a reinforced concrete building a very wet mixture is used with possibly a poor aggregate, which brings this fine material and dust and dirt to the top, and to this mushy top mixture composed mostly of the poor aggregate we try to bond another layer of material. Now we must get the bottom right before we put on the top. If you take this laitance, fine cement, etc., work it to the surface and then put a good top right on, also wet, and begin to trowel it down, it is going to draw that fine cement to the top, which causes the dusting. We must start with the top and have the mixture right, not too wet, so it will unite with the bottom, and then the top course, if of good materials, will be satisfactory.

Mr. Larned. MR. E. S. LARNED.—I would like to suggest that one of the causes of dust is the method of laying and finishing the floor, granting that poor materials, excess of water and insufficient depth are all contributing causes.

A normal cement will ordinarily require, before ready for final finish, from 3½ to 5 or 6 hours. On some recent experiments, laying some 3 ft. squares, I took the actual time the material was screeded, the time it was first trowelled, the time of the second trowelling and the time of the third trowelling, the finish. It appears to be necessary to give a cement floor, which is required to be brought to a very smooth surface, just this treatment, but I

believe in this lies a very considerable cause for dustiness. In 5½ or 6 hours, a material mixed comparatively rich like 1 : 1, 1 : 1½ or even 1 : 2, the cement has taken at least an initial set, so that the final trowelling is virtually breaking the set and rendering the surface itself incapable of ultimate hardening, and which when soft or weak results in dustiness. Mr Larned.

I think that where a finish is to be put down some time after the structural floors, the remarks of Mr. Vinton are distinctly important and should be emphasized to the architect each time. I believe it is an impossibility with a finish an inch or less to produce a satisfactory floor on an old structural concrete floor, even if extreme care be taken.

Surfacing, acid treatment, or washing, or even raking the old floor after it has taken an initial set, permits of a better bond, and furnishes a very good foundation for the finish. At the same time, however, it serves to collect dirt and dust, and if there is quite an interval between the time of laying the structural floor and the time of finishing, it is very difficult to remove the dirt.

It seems to me that some day we will have a very different system of trowelling floors. I think the hand trowel or float of 10 or 12 in. in length will be supplanted by a larger tool practically covering the full width of an ordinary 4-ft. slab. Then instead of three trowellings to bring the surface to a true plane it can be done in practically one operation, obviating the apparent injury to the cement itself by reason of trowelling after it has taken a partial set.

Mr. R. W. Hilles.—It seems to me in spite of the able suggestions made that we will have in the future a good many concrete floors that will dust. This takes us back to the original suggestion of Mr. Wason that something in the nature of boiled linseed oil will be required on floors yet to be laid. Boiled linseed oil is a pretty expensive material to-day. Mr. Hilles.

Attention should be directed to the fact that a volatile oil carrying something in the nature of wax (a volatile oil in solution with paraffine) would not cost more than 18 to 22 cts. a gallon; and, mixed 1 lb. to the gallon, would make a floor dustless, probably without the complications referred to. I have found in six different cases in the last year and a half, it has worked better than linseed oil and at much less cost.

Mr. Boynton. MR. C. W. BOYNTON.—Mr. Wason told me just a month ago that he built floors that would not dust, but when he finished a floor he did not know that it would not dust. Though he has built dustless floors, as have many others, no one knows just what must be done or avoided to get such a finish. Floors that dust can be treated successfully but what we are aiming for is to construct floors that will not dust. It is not sufficient to be able to recommend a cure for a dusting floor and treatments which will successfully stop dusting are valuable.

I cannot agree with Mr. Wason that cement when properly handled will not give a hard surface and that the wear must be taken entirely by the aggregate. I believe the cement matrix bonding the particles of aggregate together is as capable of resisting wear as the aggregate itself. If Mr. Wason's theory is correct our efforts to determine how dustless floors may be laid will be in vain, for the cement in the surface of a floor must necessarily take its proportion of wear. If the work is handled so that the full strength of the cement is developed, I am confident that its wearing qualities will be entirely satisfactory under all reasonable conditions.

Mr. Ketchin. MR. M. W. KETCHIN.—I just want to relate a little experience I had recently in putting in a long line of concrete sidewalk, part of which was up a 10 per cent. grade, which experience bears out the statement about trowelling the top dressing too much. I put down the rough concrete in the usual way and followed it up with a 1 in. top of 1:2. As this top was rather soft it began to sag after the screed, going up the 10 per cent. grade. My foreman went over this portion with the screed several times to pick up this sag. When the walk hardened and was used I found the part which had been gone over so many times dusted badly while the remainder of the walk did not.

SOME NEW METHODS IN SIDEWALK AND CURB AND GUTTER CONSTRUCTION.

By Jerome B. Landfield.*

The field of street concrete work is not a large one, but it is none the less of sufficient importance to justify attention and study. The laying of sidewalks and crossings and the building of concrete curb and gutter have not kept pace with other branches of concrete work in point of development, and there has been a general tendency to regard them as rather inferior branches requiring little technical ability or experience. The reason for this is not far to seek. With the great progress of structural concrete work the larger contractors found their hands full of work that gave them better returns with less effort, and therefore sub-let the sidewalk work that came to them, or declined it altogether. At the same time the foolish struggle for cheapness on the part of short-sighted municipalities as well as individuals, brought into the field hundreds of untrained and ignorant contractors, who undertook sidewalk work at prices far below what an intelligent and honest contractor who cared for his reputation would think of doing it for.

One result of this was the general impression that anybody could do street concrete work, and that this branch of work did not deserve the technical study that was being given to other lines. The other result is that the country, from one end to the other, abounds in poor sidewalk and curb and gutter work, large quantities of which will have to be relaid. I can point out to you several cities in which bad work has caused concrete to fall into disfavor, so that to-day natural stone is being used almost exclusively. It is up to us to change this condition of affairs, and we can best do so by improving the methods in use and by educating the public to pay a fair price for good and honest work.

It will be my endeavor to point out certain developments in sidewalk and curb and gutter work, and to make some suggestions as to the best methods to be followed. It is probable that few of

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these suggestions will be new, and it is also likely that some of them will meet with opposition, but if I shall have been able to have aided anyone to raise the quality and lower the cost of his work, I shall feel amply repaid for the effort; and if anyone differs honestly on any of the points raised, I shall welcome his discussion. What I have to say is the result of practical experience and careful study together with an extensive observation of concrete work in nearly every part of the country.

One part of my task has been made easy by the work on Standard Specifications done by the Committee of this Association on Roadways, Sidewalks and Floors. The effect of their report* has been far-reaching and has done more to improve the quality of sidewalk and curb and gutter work than most people realize. It has not only guided many city engineers in preparing specifications for municipal work, but it has suggested to many contractors improvements in methods. It was manifestly impossible for this report to go into all the details of method and to prescribe exactly how the contractor should do his work in all particulars, but as far as they went the specifications were excellent. They did not permit the contractor to let the foundation course set before putting on the top course, they provided that retempered concrete should not be used; they forbade the sprinkling of dry cement on a wet surface to dry it for quicker finishing; and they prescribed other important details.

What I have to offer for consideration are, first, the developments of the past year in the matter of new devices for simplifying and reducing the cost of sidewalk and curb and gutter work, and secondly, some improvements in method that are closely connected with the use of these devices. The chief ones of the devices are steel forms or molds to take the place of the ordinary wooden ones heretofore used, and I am free to say that it is doubtful if in the whole field of concrete work a more important invention has been put on the market. The old system of using "two-by-fours" and stakes for sidewalk work is such a clumsy and expensive one that it was high time something should be devised to replace it.

This is a problem to which not a few men have devoted themselves, and there have been several stages in the development of

*Proceedings, Vol. VI, p. 518.—ED.

steel forms. Among these were various styles of templates or division plates for use in connection with wooden side-rails for the making of expansion joints. For the purpose of this discussion I will not undertake to enumerate these, but will confine myself to the perfected invention, to all-steel forms as I have had an opportunity of observing them in practical operation.

Like many another contractor, the inventor was bothered by

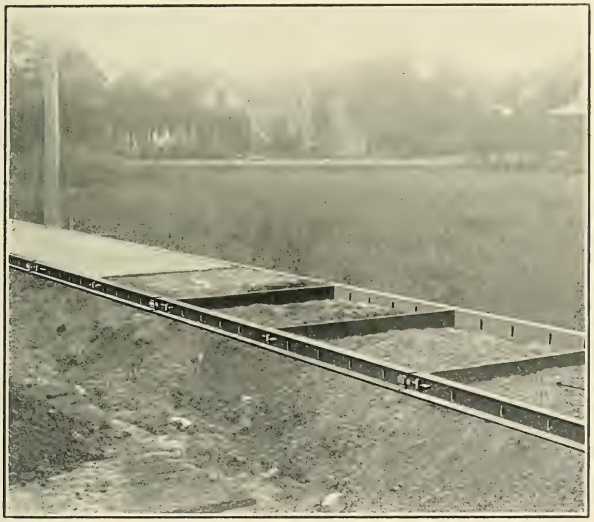

FIG. 1.—STEEL FORMS FOR SIDEWALK CONSTRUCTION.

the lumber problem and sought a means of avoiding it, not only because he objected to furnishing the neighborhood with free firewood, but also because of the unnecessary amount of labor consumed in form-setting. The question was how to make a metal form which should be strong and rigid and of which the parts should be mutually self-supporting and easily taken apart, and yet at the same time have no projections above the surface to interfere with striking off and finishing. At first the "egg-crate" form of construction was tried, using angle-irons for side-rails. These

were notched half way through on one side at intervals, with corresponding notches in the division strips to fit into them. These were found impractical because the upper edge of the side-rails was weakened by the lack of a flange and by the frequent notches, which also quickly filled with concrete. He also tried various methods of fastening the division plates to the side-rails that would permit their removal before taking away the side-rails. Was it not possible to use a form in which the side-rails should be removed first?

This was the stumbling-block and he attacked it boldly. After all, steel side-rails would be an expensive proposition if such a number were necessary, so as to leave them on the work for twenty-four hours, and besides, why should they not be removed at once? To protect the edges? Of course not, for the whole surface and not the edges alone had to be protected from intruders in any case. The idea was revolutionary, but it was worth trying, and the result proved highly satisfactory.

The idea was first applied to sidewalk work. Simple steel channels 4 in. high and having 1-in. flanges were used for side-rails, Fig. 1. At regular intervals were vertical slots for receiving the tongues of the division plates, and the ends were provided with a tongue and socket arrangement for joining successive side-rails together. The division plates were made of thin steel, 4 in. wide, and at each end was a tongue which went through a slot in the side-rail and was fastened with a lug on the outside. For different widths of walk different sets of division plates were made, but the same side-rails answered for all.

When the steel forms were placed in operation on sidewalk work, advantages appeared that were not at first thought of. Of course they did away with the annoying lumber bill and they guaranteed clear-cut expansion joints without additional expense. They saved much more in labor than had been anticipated. In the first place one man could set up as many feet in an hour as two good form-setters of wooden forms could in half a day. The measuring was already done by the regularly placed slots in the side-rails and there was no danger of mistakes. When one side was brought into line the other side had to come into line also without further work. Adjustment to grade was simple and required no changing of

stakes, and best of all the forms stay in place without any staking or bracing.

Figs. 2 and 3 show the adaptation of steel forms to intersecting and curved walks.

Before taking up the adaptation of steel forms to the building of curb and gutter, attention is called to certain points in sidewalk work that are frequently overlooked. It is not necessary to point out the great importance of the expansion joint. A great many contractors from sections of the country where there is little or no

FIG. 2.—ADAPTATION OF STEEL FORMS TO INTERSECTION WORK.

frost are inclined to regard the expansion joint as unnecessary, but this is a great mistake. It is quite as necessary to allow for expansion from heat and from the absorption of moisture, as it is to prevent the cracks due to heaving from frost, and few contractors realize how long this expansion continues before it reaches its maximum. Only a few weeks ago when I was discussing the situation with one of the best posted men in California, he said to me, "Clean-cut expansion joints would eliminate fifty per cent. of all the sidewalk trouble in this State." And yet not long ago a man who was presumably a prominent contractor in Florida—at least

his stationery led one to believe he was—wrote that he objected to clean-cut expansion joints because seeds were likely to lodge in them and weeds grow up in the walk! But of course you cannot argue with a man like that—his mind was pretty well clogged with mental weeds already. The expansion joint is a necessity, but there are so many satisfactory division plate devices now on the market that it is no longer economical to chop through the concrete; and to fill the joint with sand as a guarantee that the joint is really cut through is absurd.

The first suggestion as to the methods of sidewalk work is that altogether too much attention is paid to putting in a cinder sub-base. It may sound revolutionary, but I venture to say that in regions where there is considerable frost, in ninety-nine cases out of a hundred the sidewalks would be better if they were laid directly on the ground without any sub-base at all. The fact is that the theory of the sub-base is all wrong. If you will go out into a field on a winter day and lift up a fair-sized flat stone, you will find very little frozen ground. The natural ground beneath it hugs it closely and leaves little space for water to find its way. The same is true of a concrete flag laid directly upon ground that has been well tamped down. If, on the other hand, you dig out a trench for six or eight inches below it and fill this with cinders, it not only makes a splendid reservoir for catching the water, but the cinders act as a regular sponge to hold it, regardless of how well you may tile and drain the trench.

The next point has to do with the consistency of the concrete for sidewalk work. On this point authorities differ, especially as regards the amount of water to be used in mixing the top coat. I am inclined to believe that the worst fallacy existing in concrete work to-day is the theory that the wetter you make your mixture for sidewalk the better, especially in a warm climate. And whenever I see a man putting a top coat of the consistency of mortar on his walk I know at once that he does not know the source of most of his sidewalk troubles.

First of all it must be realized that sidewalk work differs from structural concrete work in two important respects. First it is spread out in a comparatively thin mass subject to the absorption of the ground on one side and to atmospheric influences on the

other. And secondly, the surface has to be manipulated. In structural concrete work density may be obtained by using a wet mix, but in sidewalk work all manner of harm results from too much water, especially in the top coat. First of all, if the concrete is too wet it cannot be tamped properly and the surplus water draining into the ground carries away from the aggregate particles of cement. Next, if the top coat is too wet, the neat cement and the

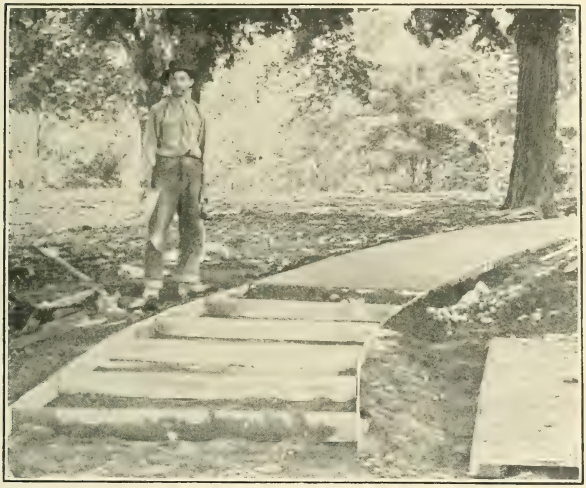

FIG. 3.—ADAPTATION OF STEEL FORMS TO CURVED WALKS.

finest particles of the aggregate have a tendency to flush to the surface and make it soft. The water that flushes to the surface delays finishing, and when the finisher does his work after delaying half an hour or so, the finer particles of cement have begun to set, and in using the trowel he disturbs them and makes poor concrete that is liable to craze. If there is more water in the top coat than the foundation course the ratio of contraction in setting is different and hair checks result.

The secret of good sidewalk work with a durable surface is

mixing the concrete with just the amount of water necessary for proper setting, and not so much that a part of it has to evaporate. Do not imagine for a moment that the top bonds better to the foundation course if it is put on wet and sloppy. To produce the best results mix the top coat stiff enough so that it can be molded in the hand and then tamp it. The idea of tamping the top coat will come as a surprise to many, but I ask you to note what the results are. In the first place an even distribution of moisture throughout the mass and a perfect mechanical bond with the foundation course are obtained. The whole mass will contract equally in setting and just enough moisture is flushed to the surface to permit of immediate finishing. How great a boon this is will occur to all who have had to pay finishers overtime for waiting around until dark for the surface to dry out. In this way the top coat is made dense and there is not, at the surface, an undue proportion of neat cement.

The tamping of a stiff top coat is closely connected with another subject of which I wish to speak, the method of finishing the surface. This I believe should consist of two operations, striking off and finishing. By striking off I do not mean the slovenly practice of slushing in a sloppy top coat and then leveling it off with a straight-edge. What I do mean is to tamp the top coat until just enough moisture flushes to the top for finishing, and then strike off the surplus material with a piece of angle-iron or a steel-shod straight-edge. For this I make a strike-off tool by taking a piece of 1½-in. angle-iron and have a blacksmith rivet a good-sized handle of round iron on either end. One man takes this at each end and together they strike off the surface with a sawing motion, keeping the surplus material on the walk ahead of them. The operation requires little time and greatly facilitates finishing. With a little practice the men become very proficient in striking off, and the result is a considerable saving in the cost of finishing. This method is hardly practicable when wooden forms are used on account of the stakes that project above the surface and in general the roughness of the lumber.

The finishing of the surface is a point on which there are many opinions. I presume that to-day a majority of sidewalk men finish their work with a trowel, but I am inclined to consider this bad practice. The great danger in all sidewalk work is too

much manipulation of the surface, and it is difficult to persuade the man with the trowel to "let well enough alone." If the strike off mentioned above has been used the simplest and best way to finish the surface is with a wooden float—on my own work I never permit a trowel to be used. Too much troweling works neat cement to the surface, and this frequently forms a smooth brittle scale over the surface that looks well for a time and then begins to chip off. Besides this troweling is likely to give a smooth glassy surface to the walk, which is exceedingly undesirable.

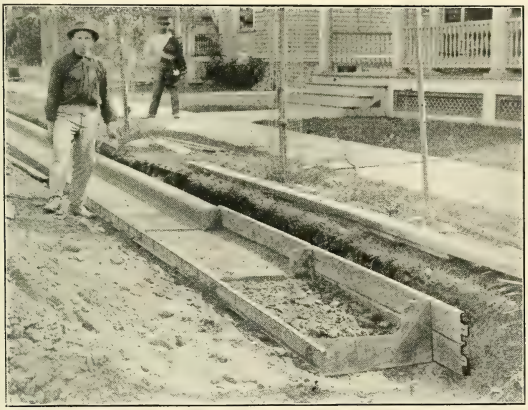

FIG. 4.—STEEL FORMS FOR A COMBINED CURB AND GUTTER.

In spite of all the progress that has been made some cities still permit the laying of sidewalk with a smooth glassy finish. Pedestrians are liable to slip on it and the general appearance is unpleasant. The better way is to finish the walk with a wooden float quickly and then brush the surface over lightly with a dampened brush. This finish is pleasing to the eye and is safe to walk on. To avoid slipping some walks are stippled, but this makes a very ugly surface and is not nearly so effective as the brushing process. In mixing the top coat it is very desirable to use a certain amount of crushed granite screenings or some similar material with the

sand in order to make a more durable surface and at the same time avoid the dirty yellowish color that common sand frequently gives. Of course it is of the highest importance to see that the sand is clean and free from loam as this is liable to cause soft spots that soon show wear.

Before leaving the subject of finishing, attention should be called to two practices that disfigure the walks in a number of cities. One of these is the custom of marking off the walk into small blocks, 18 in. to 2 ft. square. Sometimes a line is run along about 6 in. from the edge, making a sort of border. The men that have their walks laid in this manner of course have no realization of how ugly they look. The best-looking walk is that which is laid in regular squares corresponding to the width. A 5-ft. walk laid in 5-ft. square blocks without any fancy work or curlecues is handsomer than any possible variation. The point is that in sidewalk laying simplicity spells beauty. A few cities retain the clumsy custom of making blocks 2 or 2½ ft. square in the shop and then laying them, but this never makes a job that maintains its good appearance for any great length of time.

The other bad practice is one for which engineers are frequently responsible, and that is the custom of crowning the surface of the walk. Happily this practice is not very prevalent. The two main reasons why it is bad to lay a sidewalk with a crown are first that one-half the drainage is on the wrong side, and secondly that they are particularly ugly at points of intersection on account of the hollow left. A crown on a cross-walk, if not too high, is very desirable, and preferably it should be slight in the center of the walk and rather steep at each side, where the road material banks up against it. Another clever steel form has been made for the construction of cross-walks, which operates on the same principle as that already described.

There is one branch of street concrete work that is particularly interesting at the present time, and that is combined curb and gutter construction, and the recent developments in this line will no doubt arouse your admiration. The building of combined curb and gutter is of comparatively recent origin, and for its rapid development in the past few years we are largely indebted to the enterprise of the western states. It is not necessary here to point out

the numerous advantages of curb and gutter over straight curb, for the public generally is coming to realize that this form of construction is not only the most satisfactory from a practical standpoint, but that it finishes and beautifies a street to such an extent as to add more than its cost to the value of real estate. The result is that probably no branch of concrete work has shown such an increase during the past year as combined curb and gutter, and I

FIG. 5.—METHOD OF MOLDING FACE OF CURB.

venture to predict that the coming year will see three times as much work done in this line as in 1910.

For many contractors this has been a new field, in which they were without previous experience or training, and they had to get along as best they could. One may say without disparagement of their honest and painstaking efforts that their methods have been crude and clumsy. To select first-class lumber, free from warp, and to make of it cumbersome molds, held in place with a lot of

stakes driven firmly into the ground, is, to begin with, as expensive as the whole job ought to be. The fact that the wooden molds have to be left on the work for at least 24 hours means that a goodly amount of lumber is required, and very little of it is any good after it has been used three or four times. The lumber and labor involved in the building and setting of forms constituted the most important problem in curb and gutter building; improvements in mixing, in tamping, in making joints, and in finishing, were of secondary importance.

It will not be attempted to enumerate the many inventions made with a view to simplifying and cheapening curb and gutter construction—the number in the Patent Office at Washington would astonish you. The most practical were in the nature of templates to make expansion joints and to assist in holding the forms. There were also many devices designed to mold the curb by means of shaping tools, and machines for putting on the top coat and finishing. In the development of templates many excellent devices have been placed on the market, including some that permitted the construction of a "sliding grade" curb in connection with the regular gutter.

The big step, however, was made when all-steel forms were developed for curb and gutter construction. Although the plan looked all right in theory, I for one was inclined to be somewhat skeptical as to its success in practice until it had been thoroughly tested. The one point that disturbed me was the question of the curb standing up properly if the forms were removed while the concrete was still green. But this had been foreseen, and while some tests showed that with care it would stand up all right for a vertical curb, the precaution was taken to give the back of the curb a batter of 1 : 12. This not only gave the curb stability while setting, but made a stronger and better curb.

These steel forms, Fig. 4, deserve a detailed description. In general they are similar in principle to the steel sidewalk forms already described, and the sidewalk side-rails may be used in connection with them. The forms consist of steel templates shaped to a cross-section of the desired curb and gutter and a series of steel channels similar to those already described. On each end of the templates are one or more tongues which are put through the

slots in the side-rails and keyed with a lug on the other side. In this way a very strong and rigid form may be built up out of several side-rails, their flanges adding greatly to the strength. The position of the tongues on the template is determined by the number and size of the side-rails it is desired to use. The style of combined curb and gutter most generally specified is one in which the height of the curb over all is 12 in., the width of the curb, 6 in., the face of the curb, 6½ in., the width of the gutter, 18 in., the thickness of the gutter, 5 to 6 in., and the dip of the gutter toward

FIG. 6.—STEEL FORMS FOR CURBS.

the curb, 1½ in. For a template of this size it is convenient to use two 6-in. side rails on the back and one 6-in. side-rail in front; but for the man who is building sidewalk at the same time, as frequently happens on real estate development work, it is a great advantage to have the templates made to accommodate three 4-in. rails at the back and one 4-in. rail in front, so that he can use his sidewalk outfit to build curb and gutter by the addition of a few templates.

To mold the curb a face plank is clamped on the template and

the space between this and the back mold is filled with concrete and tamped. The face plank should have a layer of top coat material placed against it so that the face of the curb can be floated on removing the form without plastering. The whole surface may be struck off with a piece of angle-iron in the manner described for sidewalk work, except that successive templates are taken as guides for the strike-off tool.

The setting of the forms is a simple matter. The best method is to drive a series of stakes to a grade even with where the bottom of the forms ought to come. Then lock the forms together firmly and set them on these stakes, which makes further leveling unnecessary. They are then ready for filling with concrete. This should be mixed stiff enough so that it may be tamped without quaking. Next put on the face plank as described above and mold the curb. The face plank may be held in place by braces from the front rail instead of using clamps (Fig. 5).

The final step is to knock out the keys and remove the forms, which may be done as soon as the surface has been finished. After removing the template the groover should be used to finish the joint. It will be seen that the same forms are used over and over again the same day. With these forms I have seen 50 lineal feet used to build 300 lineal feet of curb and gutter in one day.

By making a template of a different shape and using the same side-rails it is possible to build straight concrete curb (Fig. 6). For this purpose a batter of 1: 12 is given to both front and back of the curb. A convenient size for this is a curb 16 in. high, 6 in. thick at the top and 9 in. thick at the bottom, as this permits the use of four 4-in. sidewalk side-rails front and back.

In the foregoing it has been endeavored to point out some details of method which practice and observation have shown me to be sound and good, and also to describe some of the latest and most improved devices for reducing costs. Methods and devices alike are offered for study and discussion. I shall be happy indeed if the above shall have in any degree contributed toward the object in which we are all interested, the improvement of concrete work, and the success of the conscientious, progressive, and public-spirited contractor.

TOPICAL DISCUSSION ON ROADWAYS, SIDEWALKS AND FLOORS.

Mr. C. F. MacEvoy.—Some of the floors in a building at Newark dusted. For instance: there are flags in a floor all laid the same day, where the dusting is very noticeable. Flags immediately adjacent have a good hard glassy surface. The architect blames the cement, or lack of cement, but the trouble was not with the cement. Tracing back, it was found that the cement finish had been laid over a gravel concrete placed on top of brick arches. It seems that in places the concrete did not absorb the excess of water, and after finishing there was a thin film of water and cement left on the surface. It is this film that afterwards dusts off. Mr. MacEvoy.

It is our practice to mix up a drier of even parts cement and sand to absorb this excess water, and bring the surface so that it can be troweled and so that the joints separating the surface into flags can be made without caving in. We had not discovered just what action the water, coming to the top and bringing the fine particles of cement with it, has on the cement, but the result is that if this water is not entirely taken out, the surface will dust. I do not know whether the water dries out or eventually goes down through, but it leaves this film over the top that can be brushed off. The surface of these blocks was rubbed down with a carborundum brick, possibly 1/16 in., or to the depth of the film referred to. It has been said it is the drier that dusts off, but this is not the trouble. The work is floated with a wooden float, and then troweled with a steel trowel to a smooth finish, making the drier and the cement wearing surface monolithic. I agree with those who condemn the practice of using a drier that it is attended with unsatisfactory results when a mixture of pure cement is thrown on the surface and simply troweled in. It has been said that if the finish is ground down that the floor will then present a surface not subject to dusting. This may be all right but it looks bad and if it is subjected to much walking or heavy usage, the particles of sand and cement having a sandpaper appearance, will gradually be further ground down and cause almost as much dust as before.

Mr. MacEvoy. It is really this troweled, glassy finish that forms the wearing surface, without any projections to become loosened and ground out; whereas, if this finish is defective and has to be taken off, we have the sandpaper finish mentioned with the dusting result.

If a drier is properly put on and with no excess of water, there will be no dusting. I have heard very few complaints of dusting in sidewalk construction, and attribute this to the work being done outside where the action of the sun and air evaporates and takes care of the excess water coming to the surface. Some will say that too much water is used, but I think that it is necessary to have it in order to make a pasty mortar so as to coat each of the sand particles with cement. You will find, where the cement finish has been laid one inch thick or more, over reinforced concrete floors that have been laid previously, that it will dust in most cases. Here the water cannot get away, cannot go through the rough concrete slabs. Sometimes from the floor below the water is seen dripping through as there happen to be places where some of the water can get away, but the big part remains. This water comes to the top and brings a little film with it,—and that deposit of cement coming through afterwards wears away.

We have, on a very warm day in the summer, laid a cement finish over a reinforced concrete arch by keeping the men there half the night to gradually take this water off, with this additional drier; a piece of blotting paper would do the same thing. Afterwards a sand drier was used, about a quarter of an inch thick, which dries up the water and is then taken off and thrown away. It is used simply to dry up the water the same as a piece of blotting paper.

If we would have waited for the water to evaporate or pass away naturally, it would then be too late to trowel the cement or cut the joints. I have had cases where the floor could be finished the next morning if the weather was cold, but in the summertime you could not wait that long.

Mr. Stagg. MR. CHARLES J. STAGG.—I have had quite a little experience and have laid a number of engine room and basement floors and I have not had any floor to dust yet. I have tried all sorts of experiments and put a little coloring in the concrete. The floors look very nice, and do not look like a cement floor. Of course,

we cannot wait till the next morning to rub it down as stated, but have to do it right away; make a cement floor and then make a kind of drier on top when it is wet, say 1 to 1, and then put in some finish of red color to make it pink. I use a waterproofing compound, and never had trouble with it. When I do one kitchen floor I get another contract and have done sidewalks the same way. Mr. Stagg.

Mr. J. B. Landfield.—My experience is more with sidewalks than with floors, and in this of course the question of dusting is less important. If the top coat is mixed fairly stiff, using a proportion of about 1½ to 1, and thoroughly tamped, there will result a good dense course that can be floated immediately. Only enough water should be used to supply the moisture needed for setting, and not enough so that the surface will be slushy or show water standing on it. The top coat is tamped and floated with a wooden float and there is no film of neat cement on the surface to chip off afterwards, which is particularly bad on a floor across which machinery has to be moved. Cement flushed to the surface by troweling when too much water has been used and cement sprinkled on to absorb surplus water ruin many floors and walks. Mr. Landfield.

The character of the sand has much to do with the prevention of the dusting of a concrete floor. If the sand has a little loam in it or contains some organic matter, it will wear away in spots. To make a fine wearing surface use sharp clean sand, and preferably mix some granite or trap screenings in it. Then mix it thoroughly and do not use so much water that it cannot be tamped, or so that the men have to wait around for it to dry out before finishing. If this is done, finishing will disturb the particles that have begun to crystallize and ought not to be disturbed. If the lower course has already set, it should be thoroughly cleaned and soaked, so that it will not absorb moisture from the top coat; it should then be grouted and the top coat put on as dry as possible and still have enough moisture in it to bring about setting. Tamping will bring sufficient moisture to the surface for immediate finishing. A little practice of this kind will solve many a problem.

Mr. MacEvoy.—In Newark, some 200,000 feet of cement sidewalks are laid each year. They are not laid as described by Mr. Landfield, that is, they are not manufactured like a concrete block. Mr. MacEvoy.

Mr. MacEvoy. It is believed that there must be an excess of water in the cement finish and in the concrete.

There should be enough water in the mixture to carry the cement through and to properly coat every particle of sand that goes in it. Now if it is manufactured like a concrete block, there is not the proper distribution of cement. Making the concrete and the cement finish as wet as above, it is not mixed 1½ to 1; it is not necessary. This paste mixture properly coats all of the sand with a 2 to 1 mix and gives a better result than with a dry mix. This has been demonstrated by tests. A better cement finish and a more lasting surface is obtained than with a 1 to 1 dry mixture, as it is made for blocks.

With the wet mixture it is necessary for a drier to be used. This does not at all mean a film on the top. It is not a film on the top; it is the richer mixture put on to dry the water out in the first instance and floated in possibly to a depth of ⅛ or ¼ of an inch, and to give a good wearing surface, not a glassy surface and not a pure cement surface such as is used in New York on floors. I have seen a mechanic screen neat cement over the surface and then simply take a steel trowel and trowel it. That is a film; the drier referred to is not a film at all. If a concrete slab is laid and immediately covered with a cement finish, the mass is monolithic. The top coat and drier can be explained as monolithic on the same principle.

In all of the cement finish for sidewalks, floors, etc., laid throughout New Jersey, there is no such thing as tamping. It is deposited in the forms from pails or wheelbarrows in a fluid pasty mass and brought down with a straight edge; it cannot be tamped.

Mr. Boynton. MR. C. W. BOYNTON.—I think there are two extremes in concrete work. The ordinary concrete block is one extreme and the very wet, coarse concrete is the other. I believe the intermediate way is better.

Mr. Genthner MR. WM. GENTHNER.—I would like to ask if it was not possible to mix the cement and still have the pasty appearance and not have an excess of water.

Mr. MacEvoy. MR. MACEVOY.—In that case, when troweling, the water would come to the surface. There is a natural excess of water.

Mr. Genthner. Mr. Genthner.—It all depends on the mixing. One set of men will mix a soft mortar, another gang will make a rather stiff mortar, which if stirred will bring no water to the surface at all and that is the best mortar. It is difficult to get men and even a machine, to mix mortar right through of that consistency. Such a mixture need be followed with the trowel but once.

Two years ago there was talk of laying a single coat walk. Such a walk was laid, experimentally, in front of my building, using a mixture of from 1:4, 1:5, 1:6, 1:7, 1:8 and 1:9. It is doubted if any one can detect the difference; it was simply a float finish and to-day the float marks are not worn out yet. Such a walk with a trowel finish would naturally dust away. It is believed that with a float finish a harder surface is obtained than with a trowel, unless the latter is worked so that just the excess of water comes to the top.

Mr. Landfield. Mr. Landfield.—About as many walks are ruined by too much troweling as in any other way. The specifications should state that a steel trowel is not to be used on a walk, because the workman is inclined to use it too much. Concrete must not be worked over and over, a thing which is very easily done in the case of too much water in the mixture. Too much working simply brings an undue amount of neat cement to the surface.

Mr. Guay. Mr. T. J. Guay.—My experience with dusting of floors after they are laid points a great deal to the material used. In the first place, a lot of this trouble would be overcome with a clean, sharp sand, without any vegetable matter in it. The concrete should be mixed and the floor finished as quickly as possible. The trouble in finishing is because of too much working with a steel trowel. The more it is worked the more the water keeps coming up and is drawn out. This is not necessary if the concrete is floated at the proper time.

We also found that a little black coloring matter is of quite a benefit to the floors, makes a better looking floor and then there is not quite so much dust.

Mr. Landfield. Mr. Landfield.—I have just examined some walks out in Nebraska, where it has been the custom to mix the surface coat as wet as mortar or even wetter. The glassy surface and unsightly trowel marks are evidence of how the walks were finished, and the

Mr. Landfield. flaking off of the surface in places shows the result of too much troweling. In other places hair checks show after a portion of the particles have begun to set. A glassy surface is bad; it is not good looking, and it is not safe to walk on.

On the other hand, I have been interested in Iowa in observing a lot of one-coat sidewalk. This seems to be the general practice in Iowa, especially in the larger cities. Some engineers told me that a few years ago a good deal of fraud had been perpetrated by dishonest contractors in two-coat work which was difficult to detect, especially in skimping the cement in the lower course. To avoid this they now require one-coat work, sand and cement, one to three. They mix it fairly stiff and tamp it, and finish with a wooden float. The result is a fine walk in which there is no worry about the bond, and it is hard to tell the difference between this and a well laid two-coat walk.

Mr. Genthner. MR. GENTHNER.—It seems that under the same conditions a floor should not dust any more than a walk, unless the dust outside is not so perceptible.

Mr. Boynton. MR. BOYNTON.—I believe there is a good deal in that. The conditions are not the same, the evaporation being much greater and more rapid outside. In outside work the top is usually put on quite soon after the base is laid.

Mr. Landfield. MR. LANDFIELD.—There is another case that has not I believe been mentioned, and that is the laying of a concrete walk on an iron bridge where the conditions are somewhat similar to those indoors. Here there are varying temperatures and atmospheric conditions, and the surrounding materials do not assist by holding moisture. In work of this kind it must be taken into account that a comparatively thin course of concrete is laid where there is difficulty in keeping the whole mass evenly moist while setting. It is of great advantage in this work to mix stiff and tamp well, and as soon as the intitial set has taken place to cover the whole walk with wet sand.

Mr. Schouler. MR. W. W. SCHOULER.—One reason dusting shows inside of a building rather than out is that frequently the concrete floor is laid under artificial heat, which is very detrimental to the setting of cement. Outside of that, if the cement is all right and the aggregate is fairly good, a superfluity of water will cause dusting

every time. The heavier aggregates go down and the water comes to the surface; it is drawn up by a trowel or float. It will evaporate and leave particles of concrete standing up that will knock right off without any trouble at all. It will have the appearance of a very rough floor that never was floated or troweled. Mr. Schouler.

Some years ago I laid a floor with cement in a factory so as to eliminate the dust from the wooden floor. It was more dusty than the wooden floor. The trouble was the steam heat in the room while the cement was setting. After the floor had been used for a while and perhaps 1/32 or 1/16 of an inch worn off the top, the dust stopped. That was twenty years ago and the floor is in good condition to-day, but not as smooth as it was when first finished. A superfluity of water and artificial heat is what makes good cement dust, in my opinion.

Mr. P. H. Atwood.—As previously stated in the Central West, through Iowa, single-coat work is used and this continually increasing. In the city of Cedar Rapids all sidewalks are made this way. A rather fine aggregate is used and all the sand is pumped from the river; the concrete is mixed 1 part cement to 3 parts sand and as wet as a thin mortar. The concrete is run about 4 inches thick and struck off. A few minutes after striking, moisture is standing all over it, and soon this moisture has all passed away. It has settled down through when the walk is finished and the trowel or finish marks can be seen on these walks even after severe use. Mr. Atwood.

We make walks and floors with one-coat work, mixing 1 : 4. Floors of this kind in boiler and engine rooms have never given trouble. A floor in our coal room mixed 1 : 7 was put in and we were surprised how good that floor was and is to-day.

Mr. Boynton.—Without knowing the granulometric analysis of the aggregate or at least having a sample to observe, one cannot state definitely what would constitute a proper mixture. Mr. Boynton.

Mr. Atwood.—I would add that the aggregate is screenings from a 9/16-in. screen, and I venture to say that not more than 5 per cent. would go through a 50-mesh screen. Mr. Atwood.

Mr. Boynton.—A sample of Mr. Atwood's material recently examined contains about 20 per cent. of particles retained on a ¼-in. screen and about the same amount passes a 50, therefore Mr. Boynton.

Mr. Boynton. the material is not quite as good as Mr. Atwood thinks, and certainly is not material that will produce first-class concrete when used in the proportion of 1 : 7.

Mr. Lewis. MR. M. H. LEWIS.—About ten years ago a great many column piers in the construction of a plant were made of a mixture of 1 : 4 : 8 or about 1 in 13 of cement; and in addition to having the mixture so lean, a great many very large stones were used, making cyclopean masonry. Later the piers had to be drilled to receive iron work, and the concrete having perhaps only 3 or 4 per cent. of cement, was extremely hard to drill through.

How long do materials used to stop dusting actually do so?

Mr. Boynton. MR. BOYNTON.—Sodium silicate, if properly applied, will give good results and such a treatment I believe will be permanent. Mr. Wason recommends linseed oil and has had very good success with oil, though it darkens the floor. If the discoloration which follows the use of oil is not objectionable there is no question but what it will help to stop dusting. If the treatment does not prove permanent it can be repeated as it is simple and inexpensive. A floor inclined to dust must be treated with a filler of some kind.

Mr. Benson. MR. N. D. BENSON.—I have heard of a dusty floor being fixed in a rather novel way. The floor in a dwelling had been laid in the usual way and the servant in going through the cellar dropped a bottle of milk which spread well over the floor and it was noticed that spot did not dust. The contractor, taking the cue, spread several cans of milk over the floor, which stopped the dusting.

Some time ago I hired a certain mason to execute some work for me, because the people were very much taken with the class of work that he did. The only difference I could see was that he finished up with a mixture of 1 to 1, cement and sand, the sand being so fine that it would pass an ordinary fly screen, about 16 or 14 mesh. He threw this over the whole surface and troweled it in. When the surface was of the right degree of hardness, perhaps the next day, he rubbed it down with a soft stone. It made a very smooth surface, but perhaps within a year it was all hair cracked. This case was a matter of too much cement, or largely that.

Our men have made it a practice to save some of the top

mixture and use it as a drier. We use largely 1 : 2 or 2½ top mixture, according to how the sand runs, which is very clean and sharp, usually. We have yet to find hair cracks in any of our work. Mr. Benson.

Mr. Lewis.—Quite a number of materials advertised for floor protection have a body of casein, which is an element of milk. It occurs to me, that milk would act in a similar manner. Mr. Lewis.

I know of a case in Mt. Vernon where a contractor used coloring material; it was not satisfactory and had to be cleaned off. An oily material was used, and when scrubbing commenced, the more they scrubbed the more foam would be formed. It was soon recognized that there was a fine collection of soap and the wall was covered with it.

Mr. A. C. Birnie.—I would like some information on sawdust concrete floors, the preparation, method and treatment of same. On a contract for 5000 yards the specifications called for a 1 : 2 : 2 mixture. There was a concrete floor in the building on which to lay a cinder concrete fill covered with 1 in. of 1 : 2 : 2 sawdust concrete. This was to be finished as little as possible and still secure a perfectly level surface on which was to be laid a cork carpet. The object of the sawdust concrete was to form a nailing base for the carpet. Mr. Birnie.

Before commencing work we were curious to know what results a mixture of 1 cement, 2 sawdust and 2 sand would produce, so a number of samples were made taking the proportions 1 : 2 : 2½, 1 : 2 : 1, 1 : 2 : ½, 1 : 2 : 2, and so on up to 1 : 2 : 3. We then laid other samples reducing the proportion of sand. Finally, after consultation with the architect's representative on the job, it was decided to use a 1 : 2 : 1 mixture, which in the samples was apparently all right. The first day some 260 yds. were placed and at the end of two weeks you could almost blow this away. The proportion was then reduced to 1 of cement, 2 of sand and ¾ of sawdust. This set up very satisfactorily. There were very large shrinkage cracks, but in this particular case they caused no serious trouble. The only cracks that have developed in it so far, and it has now been in about 4 months, are where the different days' work join on the screed points, no doubt due to the sawdust swelling while mixing and shrinking when dried out. There are no cracks in the concrete itself, even in a strip 12 to 14 ft. wide, and approximately 125 ft. long.

Mr. Birnie. The sawdust came from a box factory and was of hard pine, made with a very fine tooth saw such as they use in cutting box jobs. No chips in it, only good, clean sawdust.

Mr. Boynton. MR. BOYNTON.—Sawdust is used considerably with magnesite but I have not known of its being successfully used with Portland cement.

Mr. Price. MR. PRICE.—I would like to ask just what is a granolithic floor? Technically it must be a floor in which something that is in the form of granite is used as part of the aggregate.

Mr. Boynton. MR. BOYNTON.—The name was probably first given to work in which granite screenings were used as an aggregate for the top mixture, and possibly the top was finished so that the granite chips were visible. The name is now frequently applied to work which does not contain granite and often work containing granite is not referred to as granolithic, and as the word has never been defined by a technical body it would hardly be safe to use it in specifications.

Mr. Schouler. MR. SCHOULER.—Mr. Stewart, of Glasgow, Scotland, invented granolithic and sent his specifications all the way over here, and a party in New York City represented him years ago; that was the origin of granolithic pavements.

Mr. Price. MR. PRICE.—I would like just a little further information on that, the finish of it. Does it necessarily show aggregate? Does it show granite chips and must it be rubbed down?

Mr. Schouler. MR. SCHOULER.—One-quarter-inch granite is used in a mixture of about two parts granite, two parts sand and one part cement. The granite being a large aggregate, does not require much cement, and the sand used the most of the cement; consequently it was a strong mixture.

Aggregate cannot be shown without treating the floor as they do a terrazzo floor, rubbing it down; because after troweling only the cement comes to the surface and after five or six years of wear you will see the aggregate, but not before. The original granolithic did not look different than any other pavement.

Carborundum is used quite extensively in this section for making a walk not slippery. You will find it in the Hudson Terminal, the floors of the tunnel cars and in various other places.

Mr. Landfield. MR. LANDFIELD.—While in Omaha recently, I had the pleasure

of meeting Mr. John Grant, who seems to have originated the idea of the so-called "slagolithic" sidewalk. Out there they had no granite and so he crushed the slag from the smelters and found this an excellent substitute for granite in making the surface of a sidewalk. Brushing the surface lightly with a dampened brush after finishing, roughens the surface slightly and improves the walk. Mr. Landfield.

Mr. Benson.—Four or five years ago I saw a lot of fine pea pebbles used in a sidewalk and have seen it at other places to-day. Perhaps it is smaller than the pea, and more like torpedo gravel. I was wondering if that was worked any different from the granite aggregate, as it seems to have been rubbed down or else troweled so it did not get under. Mr. Benson.

Mr. Schouler.—Yes, all the sand and gravel they use here comes from Long Island, is light colored and is used like any sand. Mr. Schouler.

Mr. George C. Prince.—For six years I have laid sidewalks of various kinds in Camden, N. J., and 95 per cent. of them had no cinder foundation, and I have had no trouble whatever. The joints are cut through, clear through to the ground, the cinders omitted and better results obtained. Most of the soil is sandy with quite a good deal of clay. Mr. Prince.

Mr. Schouler.—I have had occasion in the last three or four years to lay long stretches of 4-ft. walks on various kinds of soil, as varied as can be found. There is no difference, no matter where laid, whether it was on hard pan or clay. Frost never affects a sidewalk if the concrete is mixed right and the blocks are not too large. The maximum length of the blocks should be the same as the width and the thickness as many inches as it is feet wide, but never less than 4 ins. Mr. Schouler.

Mr. S. F. Boyer.—I have used metal forms and parting strips but laid them aside last summer. They act very nicely on a narrow walk, say 2 ft., or laying a walk around a house somewhere, but in laying a 4 or 5 or even a 6-ft. walk, anything that wide, the joints will not be straight. Mr. Boyer.

Mr. MacEvoy.—In Buffalo in 1908, we were talking about sand joints and referred to a couple of miles of sidewalk for the Essex County Park Commission in Newark, where I thought it rather a hardship to have to cut joints and fill them up with sand. Mr. MacEvoy.

Mr. MacEvoy. There is a clear parting right in the sand joints. This summer they regraded the street, laid concrete curbing and changed the grade and left those blocks—it was a 6-ft. sidewalk and the blocks were 6 ft. square—some 8 in. low. A gang of Italians with wedges and 8 × 8 in. timbers pointed wedge-shape on the edge, lifted the flags up out of place to one side and filled in 8 in. more to bring it up to the grade. They relaid the walk and it looks better to-day than the day I laid it. The joints parted clean. I believe in sand joints, and lots of them.

Mr. Schouler. Mr. Schouler.—In reference to using sand, my observation has been that there is no objection to it. It is the contraction that usually breaks sidewalks, occasionally expansion. Half an inch of sand in every 4 ft. will take care of all expansion which is less than 1 in. per 100 ft. If soft mortar is poured into a groove it will fill up and the walk will be solid and will not allow expansion.

Mr. Guay. Mr. Guay.—We have in New Hampshire 3 or 4 ft. of frost, and there is very little concrete walk laid without excavating and filling in loose material. Now I would like to know whether a walk can be laid right over the natural soil under such climatic conditions?

Mr. Schouler. Mr. Schouler.—I am familiar with New Hampshire weather and would not hesitate to lay a walk right on the ground. In the coldest part of New York State there are 10 miles of 4-ft. sidewalk laid right on top of the ground at 9 cts., and it is in fine shape.

One of the oldest bluestone men in the country stated that the way to lay a bluestone is to lay it on the ground, but because a bluestone slab comes in a 3 or 4-ft. section you can excavate sufficiently to get a bed so that the flag will lie there without rocking. That does not apply to concrete. When you come up with a wheelbarrow full of mastic concrete you dump it over uneven spaces, you tamp it in there and it forms its own bed, and water will not go under that concrete to hurt it.

A 4-ft. flag was taken out during a long spell of zero weather to get at a frozen water pipe and on driving a pick into the ground to see how much frost there was under the block, the pick drove right in to the handle. That led me to believe that frost does not go underneath a slab as in exposed places. Where frost

gets behind a curb it will raise it up, but on a level space underneath the block it does not seem to affect it at all. Mr. Schouler.

Mr. Atwood.—At Lake Park, Iowa, they do not fill in at all under the walks which look good. The soil is a black, light loam. Mr. Atwood.

At Armstrong, Iowa, there is a walk laid last year with 6 in. of gravel under it. It was cut with metal forms, so it was cut clear through. Last winter the walk heaved at least 2 in. It had a filling under it. A filling is more porous than the outside ground, and if it happens to be a little low it is a good place for the water to run in and heave a walk up.

Mr. Lewis.—I received an inquiry some time ago from a gentleman connected with the schools in this city. They have very many sidewalk vaults having playgrounds underneath. All this walk was laid in large sections, about 25 ft. on a side, of reinforced concrete, the rods of the reinforcement did not run from one slab into another. During cold weather, the contraction opens up the joints between the various slabs and water goes through; they have experienced a great deal of difficulty and have tried almost every possible way to close up these cracks. I would like to know if any one has had experience in closing up such cracks. Mr. Lewis.

Mr. MacEvoy.—I had quite an experience of that kind with a garage. It was of reinforced concrete and again it was not. We built an iron frame skeleton with trusses; it was one of these hip roofs, coming to a point in the center. In the beginning we lathed the outside of the building with expanded metal and plastered inside and out, thus making a wall about 2 in. thick. Angle irons for studding were used, 16 in. center to center, and this same thing for framing across the roof. The lath was laid across the roof and the mesh kept fine enough so as to plaster a good heavy coat. The roof was divided into expansion units, these being V shaped joints very much the same as a sidewalk joint, extending from the top surface of the roof down to the iron. Well, in that way we placed two coats of plaster on the underside of the roof, more to make a finish of it than anything else and to protect the underside of the iron. The surface was troweled just as a sidewalk and the joints caulked with oakum. The joints Mr. MacEvoy.

Mr. MacEvoy. were filled up tight with plastic cement, the object being that if the cement did contract, the contraction would open up this joint and this plastic material would then settle down and fill up this joint or opening that had been created by the shrinkage.

Again, should this slab itself expand, the effect would be to push some of this material up a little above the finished surface of the roof. During a rainy season leaks were found in the joints in some two or three places. These were marked, thinking that possibly there had been a settlement or shrinkage there and that the plastic material, on account of the weather being a little cold, had not worked down and filled up the opening. During a heavy rain some two weeks later, there was just one leak, but in a different section of the roof. And the roof does not leak at all now. If shrinkage occurs in the cold weather, a little opening would appear between this plastic cement and the joint, and if it did not warm up or was not warm inside of the building, so that the plastic material would flow, then it would leak. After awhile all the expansion and contraction will have occurred, the material will find its bed and then there will be no leakage at all.

Mr. Lewis. MR. LEWIS.—I think that conditions on a roof are somewhat different from the conditions on a sidewalk; there is so much traffic on the sidewalk and it is subject to abrasion, heavy blows and many other injurious influences. The expansion on a 25 ft. slab for a summer temperature of 100 degrees and a winter temperature of zero is nearly ¼ in., and the joints would open up each winter. They tried many materials, perhaps some of the same order. The joints would be tight after repairing and all right the next summer, but the following winter they would open up again. All along underneath, the joint was obstructed with beams and iron work of different kinds, and you couldn't put in a gutter or drain.

Mr. Genthner. MR. GENTHNER.—I have been over the concrete road with Mr. McClintock in Rochester. This road consists of 2-in. cubes, some of which are grouted and some not, the former wearing best.

Mr. Boynton. MR. BOYNTON.—Mr. McClintock says they do not wear badly. In defense of the small cube he says that most any wheel will engage two of them and for that reason will not have a tendency

to catch and pull them out. The small grouted section which I saw seemed very firm. The joints between the cubes were hardly visible and the grouting was showing very little wear. The grout was swept over the pavement after it was laid with no effort to force it between the cubes. Mr. Boynton.

The mixing was done by hand and the cubes were molded in a specially made machine patterned after the concrete brick machine. Owing to the size of the cubes and the fact that it is not important that they retain true shapes, the mixture used is much wetter than the mixture used in the manufacture of brick. The cubes are placed on pallets and set on curing racks immediately after being formed. The mixture is wet as compared with the ordinary cement brick but not wet enough to settle out of shape badly. To be somewhat irregular in shape does not seem to affect the real value of the cubes.

Mr. McClintock made cubes by another method but as cubes made by this have not been tried in the road as yet it is not possible to say whether or not they will prove a success. On the whole these cubes do not appear as strong as the cubes made in the brick machine. This scheme consists of placing a platform on which is set up on edge a number of metal strips 2 in. wide bent in such shape as to form three sides of the cube molded, the other side being formed by a straight strip.

At the Rochester plant enough platform and metal strips were provided to permit of casting 50,000 cubes at a time. The concrete was mixed wet and placed in these forms, brushed down with a broom and allowed to harden. When hardened sufficiently the cubes were removed from the mold and shoveled from the platform to the stock pile.

MR. ATWOOD.—What is the comparative cost of these small cubes and the general cement pavement? Mr. Atwood.

MR. BOYNTON.—In some experimental work put in this past season the cube pavement cost 62.6 cts. per yard. A section of single coat work, put in at the same time, of one part cement, 3.35 parts bank run gravel, 6 in. thick, cost 88.7 cts., while a section of 4½-in. concrete base and 1½-in. wearing surface cost 89 cts. per yard. The cost of material and labor for this work Mr. Boynton.

Mr. Boynton. was: Cement $1.50 per bbl.; pit run gravel 60 cts. per yard; roofing gravel $1.25 per yard; screened sand $1.00 per yard; teams $5.00 per day; foreman $4.00 per day; labor 25 cts. per hour; roller $5.00 per day.

All this pavement was laid over old macadam and to bring the cubes to the same grade as the surface of the other pavement, a layer of gravel was placed over the macadam and rolled.

Mr. Genthner. MR. W. J. GENTHNER.—Before placing the top coat on a reinforced concrete floor in a building, the base was washed with an acid solution. The top coat would not adhere to the base because it was impossible to wash the acid off as the water would seep through the floor and spoil the material below. A wash of milk of lime and cement was brushed on with a broom, and this seemed to fill all the pores and stop the leakage entirely. The top coat, ¼ in. thick, then had a very good bond.

I placed a slab of 1:4 mixture over a railway store front about 21 x 40 ft., not using any waterproofing whatever because of the short span. The top coat was placed immediately after spreading the base and screeded off. It was wet and the slope ¼ in. per ft. The water seemed to work down, and it has made a rough even pavement that has never leaked one particle; the top coat is of a 1½:1 mixture about ½ in. thick. There is a bakery underneath this slab, and no water ever goes through in the heaviest rains.

Mr. Woolson. MR. IRA H. WOOLSON.—I have not seen a cinder concrete floor which was entirely waterproof. Those which have been put into test buildings are sometimes very water resisting, I ought not to say, perhaps, that some of those floors are not entirely waterproof, because as a matter of fact we had no opportunity to test them for waterproofness. I believe that if the mixture were made 1:3:5, of good clean cinders, not too much ash, there would result a floor that would be perhaps not entirely waterproof, but certainly approaching it. It should be a good moist mixture, not a sloppy mix, and well tamped in place. But I do not believe it is possible to get one that will not soak up water, that is, become damp.

Mr. Case. MR. C. J. CASE.—There are a great many men putting in stable floors in the way Mr. Genthner suggests, and as far as I know they have never been very satisfactory. We use a rather

ordinary coarse run of gravel, and on a 4 in. floor are obliged sometimes to use it as coarse as 2½ or 3 in. Once in a while a sharp shod horse will pick one of these out of the concrete, leaving a very bad hole. A ¾ or 1 in. gravel in a wet mixture of 1:3:5 or 1:2½:5 will give a very good finish to a barn floor if tamped with a corrugated tamper having a rough bottom so as to bring the fine aggregate to the top and drive the coarse stone down. I have put in some that way and they have given very good satisfaction. I do not think there is very much saved in the labor or in the material nor a better floor obtained by placing a top coat. Mr. Case.

Mr. George R. Evans.—I find that a dot roller as we use it, and as I have seen it used, in a great many instances, is applied after the top coat has been floated up and troweled and is set enough so the roller will not sink too deep, but just indent the surface. I have found in using the roller on such a surface that the tendency is to break or disintegrate the top, at least ¼ in. of the top, so that after being used a year or so this top will be entirely worn off, but owing to the dust and rain it is scarcely noticeable. I think it is due to the roller breaking the top after it has partly set, so that it does not unite again. Mr. Evans.

In order to make the impression the material immediately between the points must be compressed. The point of the roller does not drive the top down, it simply raises the small blocks between the indentations and destroys the bond. Some men use a lighter roller than others, and necessarily must roll the surface while it is softer in order to get the indentations, which does not destroy the bond. The result is that the top is in better condition for wearing.

Mr. George C. Prince.—I have had occasion several times to endeavor to keep the water out in places where people have had wet cellars, also in constructing boiler rooms down in the ground where water frequently rises as high as 18 or 12 in. of the top of the ground, and have always used first a layer of concrete and then a layer of pitch. In some cases as high as three coats of tar paper were used, each one laid with hot pitch swept right on it, so as to make a perfectly watertight layer, and then a 2 in. layer of stone concrete with a ¾ in. top coating over that. So far I have not Mr. Prince.

Mr. Prince. failed to keep the water out, even though it was laid below the natural water level. In several cases where the water pressure was considerable I have reinforced it the same as a floor slab, in order to keep the water from pressing the wall in.

Mr. Ham. MR. W. H. HAM.—Mastic has been used many times on top of a concrete floor and the following method has been used: Take the concrete surface, finish with a wood float or a cork float, leave as rough as would naturally be done with a cork float, once or twice over; apply on the top of that a material composed largely of sand aggregates, grading the sand, and the plastic material being one of the asphalts in small proportion, and the material used very thin. In that way the objection to the extreme softness of a mastic floor was overcome.

Mr. Stagg. MR. C. J. STAGG.—I have had experience with waterproofing where there were springs in the ground and it was very hard to get a good bottom. Three or four inches of concrete were put on the bottom and then about $\frac{3}{8}$ in. pitch, pressed back to the tar paper. As the paper would in time probably wear out, a layer of waterproofing mixed with cement and sand, 1 to 3, was placed on top. It has been down 5 years and there has been no complaint. The floor was finished with a cork float.

Mr. Woolson. MR. WOOLSON.—I have tested a great many of those fibre floors and they have excelsior, sawdust, macerated wood, cocoanut fibre and every possible conceivable binder that you can think of. The claim is always made "thoroughly fireproof and absolutely waterproof." The first claim is usually substantiated; the second claim, so far as I know, has never been substantiated. If the surface with which they finish, which is a polished surface, is rubbed off,—it usually would be after a certain period of wear,—and then water comes in contact with that surface it will absorb usually as much as 5 or 10 per cent. The more wood used the more the absorption is bound to be. A floor of that character as delivered by the maker, and as it would be with moderate wear, would stand a large amount of water before there would be any serious absorption. They sometimes finish with wax or oil or some other material that would give it a waterproof surface. They make very pretty floors, but some of them absorb water enough so that they bulge and

crack. There have been numerous cases in this city where floors have been taken out of hospitals and other large buildings, that have failed, due to buckling caused by the absorption of water. There are some floors on the market at the present time that do not use wood in their composition and are giving very good results. I do not know just how waterproof these are. Mr. Woolson.

REPORT OF THE COMMITTEE ON BUILDING BLOCKS AND CEMENT PRODUCTS.

The members of the Committee do not feel competent to suggest any further changes in the Standard Specifications for Cement Hollow Building Blocks already adopted. However, they would be pleased to consider any suggestions.

The Proposed Specifications for Plain Concrete Drain Tile, which was referred back to the Committee at the last Convention, has, in accordance with instructions, been taken up with Mr. P. H. Atwood, of the Interstate Tile Manufacturers' Association, who is also a member of this Committee, and a revised specification is herewith submitted.

The proposed specifications for Architectural Concrete Blocks were also referred back to the Committee for further study and collection of data, and the Committee submits, inasmuch as the title and substance of this proposed specification does not sufficiently distinguish between the subject it is intended to cover, and that of Concrete Building Blocks, for which specifications have already been adopted, a revised title and specification. As concrete stone is made by various processes, each producing different results, it is difficult to draft a specification broad enough to cover them all, and this specification has been confined to the general requirements as to quality of material, methods of mixing, reinforcement, curing and handling, and a general requirement for strength and density, which should be applicable to concrete stone made by any process.

Respectfully submitted,

P. S. Hudson, *Chairman.*
C. K. Arp,
P. H. Atwood,
John Morlan,
Chas. D. Watson.

Note.—The specifications for Architectural Concrete Blocks and Concrete Drain Pipe and Tile appear in the following pages in their proposed form. The convention referred the specifications back to the committee with instructions to report at the next convention.—Ed.

PROPOSED STANDARD SPECIFICATIONS FOR CONCRETE DRAIN PIPE AND TILE.*

Materials.

1. Cement.

The cement shall meet the requirements of the Standard Specifications for Portland Cement of the American Society for Testing Materials, and adopted by this Association (Standard No. 1).

2. Fine aggregate.

Fine aggregate shall consist of sand, crushed stone, or gravel screenings, graded from fine to coarse, passing when dry a screen having one-quarter (¼) inch diameter holes, shall be preferably of silicious materials, clean, coarse, free from loam or other deleterious matter; not more than twenty (20) per cent. shall pass a sieve having fifty (50) meshes per linear inch, and not more than five (5) per cent. shall pass a sieve having one hundred (100) meshes per linear inch.

Mortar composed of one (1) part Portland cement and three (3) parts fine aggregate by weight, when made into briquettes shall show a tensile strength at least equal to the strength of one to three (1 : 3) mortar of the same consistency made with the same cement and standard Ottawa sand, according to standard methods.

3. Coarse aggregate.

The coarse aggregate shall consist of gravel, crushed stone, or similar material graded in size, which is retained on a screen having one-quarter (¼) inch diameter holes and shall not exceed in largest dimensions one-half the wall thickness of the pipe or tile. Aggregate containing soft, flat or elongated particles shall be excluded.

4. Water.

The water shall be clean and free from oils, acids, alkalies or vegetable matter.

Proportions.

5. Proportions

A sack of Portland cement weighing ninety-four (94) pounds net (¼ bbl.) shall be considered as one (1) cubic foot. Whenever possible the concrete shall be proportioned and mixed in quantities requiring one or more full sacks of cement. When a fractional batch is necessary the proportion of cement shall be weighed,

* Referred back to committee for further study.—Ed.

measuring in bulk being prohibited. Any device for mechanically proportioning the materials is prohibited.

6. Machine-made Pipe and Tile.

Ten (10) inches and less in diameter. The mortar to be used in the manufacture of pipe and tile 10 in. and less in diameter shall consist of one (1) part Portland cement and not to exceed three (3) parts of fine aggregate (1:3).

Twelve (12) to sixteen (16) inches in diameter. The concrete or mortar to be used in the manufacture of pipe and tile 12 to 16 in. in diameter shall consist of one (1) part Portland cement, not to exceed two and one-half ($2\frac{1}{2}$) parts fine aggregate and not to exceed four (4) parts coarse aggregate (1:$2\frac{1}{2}$:4). When the coarse aggregate is omitted the proportion shall not exceed three (3) parts of fine aggregate (1:3).

Eighteen (18) to thirty (30) inches in diameter. The concrete to be used in the manufacture of pipe and tile 18 to 30 in. in diameter shall consist of one (1) part Portland cement, not to exceed two and one-half ($2\frac{1}{2}$) parts fine aggregate and not to exceed four (4) parts coarse aggregate (1:$2\frac{1}{2}$:4).

7. Hand-made Pipe and Tile.

Ten (10) inches and less in diameter. The mortar to be used in the manufacture of pipe and tile 10 in. and less in diameter shall consist of one (1) part Portland cement and not to exceed three (3) parts fine aggregate (1:3).

Twelve (12) to sixteen (16) inches in diameter. The concrete or mortar to be used in the manufacture of pipe and tile 12 to 16 in. in diameter shall consist of one (1) part Portland cement, not to exceed three (3) parts fine aggregate and not to exceed five (5) parts coarse aggregate (1:3:5). When the coarse aggregate is omitted the proportion shall not exceed three (3) parts fine aggregate (1:3).

Eighteen (18) to thirty (30) inches in diameter. The concrete to be used in the manufacture of pipe and tile 18 to 30 in. in diameter shall consist of one (1) part Portland cement, not to exceed three (3) parts fine aggregate and not to exceed five (5) parts coarse aggregate (1:3:5).

Mixing.

8. Mixing.

The ingredients of concrete shall be thoroughly mixed dry, sufficient water added to obtain the desired consistency, and the

mixing shall continue until the cement is uniformly distributed and the mass is uniform in color and homogeneous.

When the conditions will permit, a machine mixer of a type which insures the proper mixing of the materials shall be used. 9. Machine mixing.

When it is necessary to mix by hand, the mixing shall be on a watertight platform and shall be done as follows. The fine aggregate shall be spread evenly upon the platform, then the cement upon the fine aggregate, and these mixed thoroughly until of an even color. The water necessary to make a thin mortar shall then be added and the mortar spread again. The coarse aggregate, if dry, shall first be thoroughly wetted, shall then be added to the mortar, and the mass turned with shovels or hoes until thoroughly mixed and all of the aggregate covered with mortar. At the option of the operator the coarse aggregate may be added before, instead of after, adding the water. 10. Hand mixing.

Machine-made Pipe and Tile. For machine-made pipe and tile the concrete shall be as wet as can be used and allow the immediate removal of the jackets. The concrete must be used within twenty (20) minutes after the time it is mixed. 11. Consistency.

Hand-made Pipe and Tile. For hand-made pipe and tile the consistency of the concrete shall be such that it will be forced into every part of the mold by jarring or tapping. The concrete must be used within forty (40) minutes after the time it is mixed. The use of concrete requiring tamping is prohibited.

Retempering mortar or concrete, *i. e.*, remixing with water after it has partially hardened, shall not be permitted. 12. Retempering.

Forming.

Under no circumstances shall pipe or tile be made in a temperature lower than forty (40) degrees Fahrenheit. Pipe and tile shall be formed so as to prevent laminations or planes of weakness. 13. Forming.

Machine-made Pipe and Tile. The pipe and tile shall be made in such a manner as to insure a dense and uniformly packed product with smooth ends and inner surfaces and show water marks on the outer surface (web-like markings indicating free moisture).

Hand-made Pipe and Tile. Concrete for hand-made pipe and tile shall be forced into the metal forms by jarring or tapping in

such a manner as will obtain a smooth, dense surface and shall remain undisturbed in the form for at least twelve (12) hours.

Curing.

14. Natural curing. Plain concrete drain pipe and tile shall be protected from wind current, direct sunlight and dry heat for at least six (6) days and from frost for at least twenty-one (21) days.

Warm weather. After hardening sufficiently the pipe and tile shall be sprinkled at least three (3) times a day for a period of six (6) days, after which they shall be allowed to cure at least twenty-four (24) days, being sprinkled three (3) times a day for the first six (6) days.

Freezing weather. After hardening sufficiently the pipe and tile shall be sprinkled at least three (3) times a day for nineteen (19) days, and allowed to remain two (2) days without sprinkling. During the time of curing, the pipe and tile shall not be subjected to a temperature of less than forty (40) degrees Fahrenheit.

15. Steam curing. The steam curing chamber shall contain an atmosphere saturated with moisture and maintained at a temperature between seventy (70) degrees and one hundred and twenty (120) degrees Fahrenheit. The steam shall be admitted through a perforated pipe placed in the bottom of a trough, containing water, at least six (6) inches wide by six (6) inches deep, extending the full length of the chamber. The perforations of said pipe shall be on the bottom, and the surface of the chamber floor shall slope toward and be flush with the top of the trough.

Warm weather. The pipe and tile shall be removed from the molds as soon as conditions will permit and subjected to steam curing for at least forty-eight (48) hours, removed and allowed to cure fourteen (14) days, being sprinkled at least three (3) times a day for the first seven (7) days.

Freezing weather. The pipe and tile shall be removed from the molds as soon as conditions will permit, and subjected to steam curing for at least ninety-six (96) hours; removed and sprinkled at least three (3) times a day for ten (10) days, and then allowed to remain two (2) days without sprinkling. During the time of curing, they shall not be subjected to a temperature of less than forty (40) degrees Fahrenheit. At the option of the manufacturer the pipe and tile may be subjected to steam curing for seven (7) days and used without further treatment.

Dimensions.

16. Dimensions.

Diameter. The diameter or size of pipe or tile shall refer to the inside diameter and be uniform in all directions. Tile with a diameter of more than thirty (30) inches shall be reinforced.

Thickness. The thickness of the pipe or tile wall shall be uniform throughout and shall not be less than one-tenth (1/10) of the diameter, with a minimum thickness of one-half (½) inch.

Length. The length of pipe or tile shall be uniform at all points and not less than the diameter, with a minimum of twelve (12) inches and a maximum of thirty (30) inches.

Finished Tile.

17. Finished Pipe and Tile.

Finished pipe and tile shall be free from cracks and other defects which will appreciably diminish their strength, and shall give a clear metallic ring when struck with another tile or with a hammer.

PROPOSED STANDARD SPECIFICATIONS FOR ARCHITECTURAL CONCRETE STONE.

General.

This specification is intended to cover the general requirements for quality of material and products for the manufacture of Concrete Stone, commonly termed Artificial Stone, by the two generally accepted methods known as the WET and DRY process.

Materials.

1. Cement.

Cement. The cement shall meet the requirements of the Standard Specifications for Portland Cement of the American Society for Testing Materials, and adopted by this Association (Standard No. 1).

2. Aggregates

Aggregates. The aggregates shall be of clean, hard, durable material, and shall have no coating of clay or other substances which would in any way interfere with the bond between the cement and the aggregate.

The fine aggregate shall consist of sand or crushed stone, graded from fine to coarse, but all of it passing when dry a screen having ¼ inch holes; not more than twenty (20) per cent. shall pass a sieve having fifty (50) meshes per linear inch, and not more than six (6) per cent. of it passing a sieve having one hundred (100) meshes per linear inch. It shall be free from vegetable loam or other deleterious matter. Briquettes made with this fine aggregate and the cement to be used, in the proportion of 1 to 3, in conformity with the Standard Specifications, shall show a tensile strength of not less than 150 lbs. per sq. in. at seven (7) days, and not less than 200 lbs. at twenty-eight (28) days.

The coarse aggregate shall consist of gravel or crushed stone, free from deleterious materials; it shall be graded in size, but all of it must be retained on a screen having holes ¼ inch in diameter. In no case shall the maximum dimension be greater than one-half the minimum width of any section of the finished stone.

Aggregates shall be so graded and mixed in such proportions that when made into three (3) inch cubes will meet the require-

ments for strength in compression and density as hereinafter specified.

Coloring Matter. Where color is required only the most permanent and durable mineral colors shall be used, and shall be considered as aggregate.

Water shall be clean, free from oil, acid, strong alkalies or vegetable matter. 3. Water.

PROPORTIONS.

The proportions of cement to aggregate shall be such as to require the minimum amount of cement to produce strength and density as hereinafter specified. The proportions of the various sizes of aggregates and cement shall preferably be made by weight. If by volume, a bag of Portland cement weighing 94 lbs. shall be considered one (1) cubic foot, and such mixtures must be prepared in quantities requiring one or more full bags of cement. 4. Proportions.

MIXING.

The ingredients of concrete shall be thoroughly mixed dry, sufficient water added to obtain the desired consistency, and the mixing shall continue until the cement is uniformly distributed and the mass is uniform in color and homogeneous. 5. Mixing.

(*a*) *Measuring Proportions.* Methods of measurement of the proportions of the various ingredients, including the water, shall be used which will secure separate uniform measurements at all times.

(*b*) *Machine Mixing.* When the conditions will permit, a machine mixer of a type which insures the proper mixing of the materials throughout the mass shall be used, but the use of any device for mechanically proportioning the materials, except by automatically weighing, is prohibited.

(*c*) *Hand Mixing.* When it is necessary to mix by hand, the mixing shall be on a watertight platform and the materials shall be turned until they are homogeneous in appearance and color.

(*d*) *Consistency.* For *Dry Process* the material shall be mixed with the maximum amount of water permissible, and must have sufficient water so that the mixture will hold its form after being compressed in the hand.

For *Wet Process* the concrete must have at least a sufficient

amount of water to make it so soft that it must be handled quickly to prevent it running off the shovel, but not so thin as to cause segregation of the aggregate.

6. Retempering.

(*e*) *Retempering.* Retempering mortar or concrete, *i. e.,* remixing with water after it has partially hardened, shall not be permitted.

REINFORCEMENT.

7. Reinforcement.

All lintels, bearing stones, and other stone subjected to cross bending shall be reinforced by means of steel rods placed from 1 to 2 ins. from the tension surface and the total sectional area of the reinforcement shall be equal to one-half of one (½) per cent. of the cross sectional area of the concrete in the member reinforced. When any cast exceeds in any dimension eight times the least dimension, it shall be reinforced to insure safety in handling.

CURING.

8. Natural curing.

Natural Curing. For the purpose of securing proper curing, the casts shall be protected from the sun and strong currents of air, shall be sprinkled at such regular intervals as necessary to prevent drying, and such other precautions taken as to enable the final set to take place under the most favorable conditions.

For *Wet Process* the casts shall be cured for at least ten (10) days when seasoned in a temperature averaging seventy (70) degrees Fahrenheit, and one day more for each 5 degrees drop in the average temperature during the curing period.

For *Dry Process* the casts shall be cured for at least twenty (20) days, seven days of which time the product must be protected from the weather, and in a temperature not below forty (40) degrees Fahrenheit.

9. Steam curing.

Steam Curing. The casts shall be removed from the molds as soon as the conditions will permit and shall be placed in an atmosphere of steam saturated with moisture for a period of at least forty-eight (48) hours. The casts shall then be removed and stored for at least ten (10) days, if made by the dry process, or five (5) days if made by the wet process, being sprinkled three times a day during the first five days, when made by the dry process.

During steam curing care must be taken to maintain the temperature at not less than seventy (70) degrees Fahrenheit.

Finishing.

10. Finishing. Casts after proper seasoning may have exposed surfaces treated by any of the various methods as proposed by this Association in Specifications for Treatment of Concrete Surfaces. All surfaces and arrises of stone must be true and without imperfections, and no stone will be accepted that is chipped or marred.

Handling and Setting.

11. Handling. All stone shall be handled with utmost care. When transported and subject to rough handling, it shall be crated or packed in non-staining material in such a way as to assure no damage from chipping or abrasion. The stone shall be neatly stacked at the building site in a manner to give the greatest protection to the finished surfaces.

12. Setting. All casts having beds less than eight (8) inches in the wall shall be provided with metal bonds for the purpose of tying to the masonry backing and with hooks for lifting, which shall be cast in the stone.

Cleaning.

13. Cleaning. Cleaning shall preferably be done without the use of acid. Should it be necessary to use acid, its strength must not exceed ten (10) per cent. solution and must be immediately removed.

Variation in color and efflorescence. All blocks shall be uniform in color and free from efflorescence.

Strength and Density.

14. Strength Density. Concrete building stone shall be of such a quality that it will pass a test at the age of twenty-eight (28) days of at least 1500 lbs. per sq. in. in compression on 3 in. cubes, and shall not have an absorption to exceed six (6) per cent. measured by weight when thoroughly dried and immersed in water forty-eight (48) hours. The use of waterproofing compounds is prohibited in making test samples.

CEMENT DRAIN TILE PLANT—LAYOUT AND OPERATION.

By C. M. Powell.*

Although Portland cement pipe and tile have been made and used in this country for about half a century, plants for making cement drain tile, as we know them to-day, have been in existence only a few years. Up to about 1890 the concrete was packed into the molds by hand, but in that year this operation was greatly simplified by the invention of a machine which packed the material into the molds mechanically. About 1905, when land drainage was receiving considerable attention and its possibilities and advantages began to be appreciated, there was put on the market a machine capable of producing the smaller sizes of tile economically in sufficient numbers to allow of their extensive use for land drainage and a number of factories were soon established. In many instances these plants were built and operated by those not only inexperienced in the use of the new tile machinery and in the making of cement products, but also unfamiliar with factory management and economy. Thus the manufacture of cement drain tile was at the start a haphazard proposition. However, there has been a rapid development of the industry, due in part to improved machinery, but more especially to a better knowledge of the essential features in the methods of manufacturing cement products which have been profitably applied to making tile, and to better management resulting from experience. Although there has been great improvement, the possibilities of further improvement are by no means limited; in fact the development has only now reached a stage where in the light of previous experience, what may be recognized as good practice, is assuming definite lines.

The efficiency of a plant laid out, equipped and operated to the best advantage, according to our present knowledge of the business,

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would be very much greater than the efficiency of the majority of tile factories in operation to-day. The most important items which claim attention in considering the establishment of such a plant, are the amount of land needed and its location; the size and kind of building; provision for cement and sand storage; water system; machinery for handling and mixing the materials and making the tile; power for operating and facilities for handling the finished product.

The space needed for the plant will depend somewhat upon the stock kept on hand. This usually varies at different times of the year from practically nothing to perhaps 300,000 tile of all sizes; stocks of 250,000 of the smaller size are not uncommon in the dull season with plants doing a good business. Usually from three to five acres of ground is ample, and if not available, it is possible to get along with less without serious inconvenience. The location of the site is perhaps more important than its extent. Close proximity to a railroad, and a private siding are very desirable as it is thereby possible to get materials, at least the cement, delivered to the plant cheaper; besides a much wider market for the product is open when the tile can be loaded directly from the yard into the cars. Some factories have in this way opened up a good dealer-trade which they have found profitable. Better facilities for shipping large size tile are also obtained, making it possible to secure large drainage contracts.

The size and arrangement of the building will depend very much upon the layout adopted. The newer plants manufacturing all sizes of tile have a total floor space of from 9,000 to 15,000 sq. ft., all of which is practically on the ground level, as the nature of the materials and the process of manufacture do not seem to be adapted to buildings of more than one story.

The kind of construction should be at once durable, fireproof, economical and easily heated in cold weather. That which immediately suggests itself and seems best adapted is a building with steel roof trusses and concrete block walls with reinforced concrete for the sand bins and roofs of the curing rooms. A frame building may be cheaper in first cost but would be neither durable nor fireproof, and hence would not be economical. On first thought it may seem superfluous to make a cement drain tile factory fireproof,

but several instances may be cited where plants housed in frame buildings that have taken fire have been completely destroyed.

There are few localities where sand and cement storage would not be required; a certain amount is always necessary to guard against a possible shut-down due to running out of material. A water system is required to furnish water for the boiler, for mixing materials and for curing. Screens and elevators or conveyers for taking care of the sand and gravel with the least amount of labor will prove a large saving in operating expenses.

Tile machines are of two classes, those which make tile 12 ins. long in sizes below 16 ins. in diameter and those which make the larger sizes up to and including those 36 ins. in diameter in either 2 or 2½ ft. lengths. The longer length of large tile is preferable since a tile 30 ins. long can be carted from the machine and the jackets removed as easily as those 6 ins. shorter and the additional time required for the machine to make the extra length is not one-quarter of that required to make the same size 2 ft. long, because one-fifth of the time consumed in taking the tile from the machine and replacing the jackets is saved.

Machines for making the large tile have not been in use as long as the others but are capable of turning out a uniform product much cheaper than can be done by hand. The larger tile are more profitable than the smaller ones and plants where both machines are in operation are certainly the big money makers.

As is the case with most new machines, the strength of the different parts in the earlier models of both machines was not well balanced, but the makers have strengthened the weak parts from time to time so that now the machines stand the wear and tear very well. However, it cannot be said that they are yet perfect and no one realizes this better than the manufacturers of the machines, as is evidenced by the improvements which are constantly being made.

As regards power, it has been found that steam power gives the best satifaction because it is steady, reliable and the rated horse power can be depended upon. Since the adoption of steam curing necessitates a boiler, the steam engine is without doubt the most economical. The horse-power of both the boiler and engine installed should be sufficient to furnish ample power at all times, and in the long-run it will prove more economical to have a little

more power than is necessary rather than not quite enough, otherwise the output will be reduced. A 25 to 30 H. P. engine and two 40 H. P. boilers will be ample.

The car system for handling the smaller sizes of tile is the most satisfactory and economical. The number of cars required is naturally dependent upon the production and the length of time allowed for curing. With the time now usually given to curing 100 cars is the average number used for one machine. The standard car has a 2-ft. gauge and platform 40 ins. wide and 8 ft. long. These cars are equipped with three decks, though four decks are sometimes used when the capacity of the curing chambers is limited. The disadvantage of the four deck car is that the tile on the top deck receive too much shaking or jarring which increases the breakage materially. The upper decks are supported on a frame and hinged so as to be swung into a vertical position at one side of the car while the deck immediately below is being filled. The decks should be strong enough to hold the weight of all the tile which can be placed upon them without any appreciable deflection and should be constructed so as to give an even surface on which to set the tile. They are usually made with a small angle iron frame and a wooden deck of 1-in. lumber laid crossways of the car. The structural steel used in the construction should not be too light as thin shapes soon rust under the conditions which exist in the curing chambers, weakening the car and shortening its life.

Cars without springs are much used and give good satisfaction provided the track is well laid and the rails kept clean and free from sand, mortar and the like. However, springs may be used to take up the jolting of the car but should be strong enough to allow the men removing the jackets to step on the cars without jarring down the tile.

The capacity of a standard 3-deck car for the different size tile, is about as follows:

4 inch	344 tile
5 inch	244 tile
6 inch	172 tile
7 inch	112 tile
8 inch	94 tile
10 inch	54 tile
12 inch	49 tile

At least two transfer tables are required to handle the cars to and from the curing rooms. In handling the empties the use of switches where possible, will save labor in operation.

The object to keep in mind in the layout and equipment of a plant is that which will give the largest production of the best tile at a minimum cost for labor, materials and repairs, with the smallest investment consistent with good construction and equipment. The relationship of one part of the plant to another should be such that the various operations of manufacture can be properly accomplished with ease and dispatch.

Innumerable layouts are possible in any manufacturing establishment and one seldom finds two exactly alike. In suggesting a tile plant layout it is the intention to combine the good points

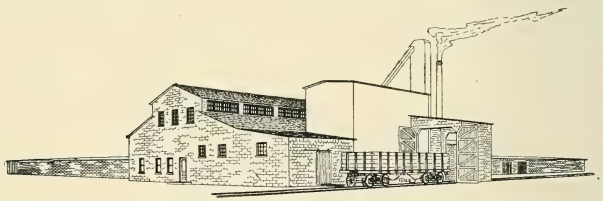

FIG. 1.—PERSPECTIVE VIEW OF MODEL PLANT FOR THE MANUFACTURE OF CEMENT DRAIN TILE.

of the various plants and to introduce a few new principles and ideas, which it is thought will result in increased efficiency. It is believed that the layout suggested is one which can be operated the year round with the minimum amount of help, and produce a maximum output of the best quality at the smallest cost. The type of construction recommended is the one which seems best adapted to the business, though it requires a somewhat larger investment than the tile manufacturer has put into his plant up to this time, but it is in common use for other classes of factories. There was a time when it was thought that the investment of a few hundred dollars was sufficient to house a tile plant but that time is passing; the tendency to-day is to build well and operate the plant on business principles.

The main part of the building, Fig. 1, is 50 by 75 ft., and in this are located the office and tile making machines. The sand,

gravel, coal and cement storage, engine and boiler rooms are in the lean-to between the main part of the building and the railroad siding. The curing chambers for the small tile are on the opposite side of the building and those for the large tile are in the rear. A row of windows placed under the eaves together with the windows in the gable ends will furnish sufficient light.

The walls of the building are made of concrete blocks with the exception of the sand and gravel bins which are monolithic concrete reinforced. The roof of the lean-to and the roofs of the steam curing chambers are also of reinforced concrete. This type of roof has been selected as embodying more good qualities than any other. Owing to the conditions existing in the chambers, timber roofs last but a few years; besides, they are not fireproof. The roof of the main building is carried upon steel trusses which are supported by two rows of columns and the side walls. A concrete tile roof, although a little more expensive than some of the other kinds of roofs, should prove very satisfactory and last indefinitely.

The plan, Fig. 2, and the sectional elevations, Fig. 3, show the interior arrangement of the plant. The mixing equipment is located directly in front of the sand and gravel bins between the two tile machines so that concrete may be delivered to either one or both of the machines. The small tile as made are placed on a car at the side of the machine and removed, a carload at a time, over the transfer track to any of the curing chambers. The curing chambers open onto a transfer track inside the building at the opposite end and by means of a transfer table the cars can be taken to the yard through the doors at the end of the track or through doors in the end of the building. The empty cars may be returned to the machine by running them around on the track at the end of the yard or by being transferred.

The large tile are conveyed by two-wheeled carts directly from the machine to the curing chambers provided for them and are removed to the adjoining cars. As it is hardly possible to locate both the large and small tile storage yards adjacent to the railroad track, it is advisable where only one siding is available, to store the large tile nearest the track, as the small tile are easily handled and most of them are sold to local trade.

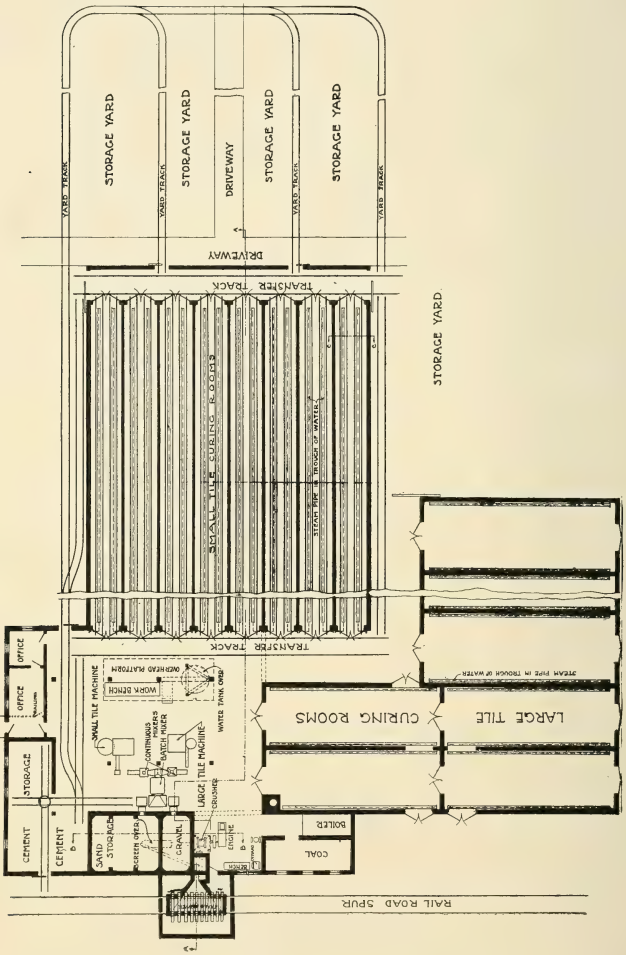

FIG. 2.—PLAN OF MODEL CEMENT DRAIN TILE PLANT.

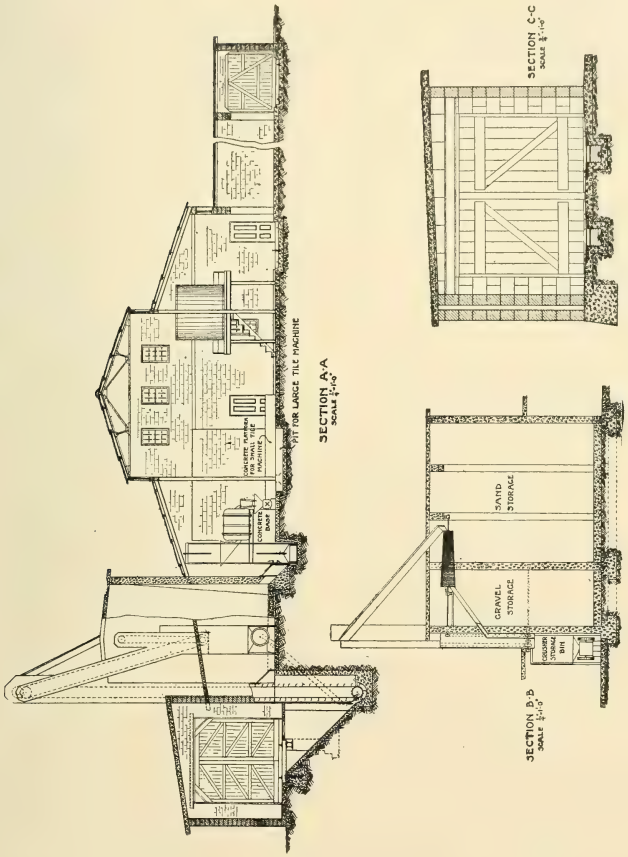

FIG. 3.—SECTIONAL ELEVATIONS OF MODEL CEMENT DRAIN TILE PLANT.

The entrance to the plant is located so that any one hauling tile must necessarily pass the office. The front entrance to the building, through which all help and others should be required to pass, is through the office, which facilitates time-keeping, etc.

In the main room an overhead platform provides a place for the extra jackets, packer heads, repairs and other accessories not in continual use and may thus be gotten out of the way. A work bench around which there is enough space available for making such repairs as can be made at the plant, is placed directly under the overhead platform. The boiler and engine rooms are placed next to each other so that one man can be fireman and engineer. A dynamo is installed in the engine room to furnish light for the plant. A small steam engine is provided to run the dynamo as lighting is required mostly at night when the large engine would not be running.

All the materials are delivered along one side of the plant preferably by cars. The respective storage bins are separated from each other as much as possible to allow of "spotting" more than one car on the track at a time. The gravel is delivered by car or by wagon and dumped directly into the track hopper from which it is elevated to the screen over the sand and gravel bins. A gravel bin is provided because it is not economical to make all the sizes of tile with the same aggregate. The sloping sides and bottom of the track hopper deliver all the material to the elevator which carries the material high enough to spout it over the top of the storage bins to the rotary screen. That part of the material smaller than $\frac{1}{4}$ in. in diameter passes through the screen and into the sand bin while that larger than $\frac{1}{4}$ in. and less than $\frac{1}{2}$ in. in diameter falls into the gravel bin.

All the coarser material is discharged from the lower end of the screen into a chute leading to a small hopper over the crusher, which is placed at the end of the bin. The crusher is set to reduce the materials to about $\frac{1}{2}$ in. and discharges the crushed gravel into the elevator which returns it to the screen. Wet gravel cannot be crushed because it will "ball" up the jaws of the crusher.

The bottoms of the bins are not hoppered or elevated which allows of additional storage capacity and the "dead" supply may be shoveled in case the material is required. The aggregates are

drawn from the bins through an opening in the side at the floor line into a measuring chute. The flow is controlled by an under-cut gate placed at the opening in the bin. When the measuring chute is filled the under-cut gate is closed and the charge is allowed to pour out into the loading skip by opening the gate in the lower end. The cement for the batch is placed in the skip with the sand which is then hoisted and the materials dumped into the hopper of the batch mixer where they are mixed dry. The mixed dry batch is then discharged into a hopper which feeds uniformly into a pug for wet mixing, and as the feed to the pug is practically uniform, the consistency of the mixture can be easily controlled. One man should be able to run the batch mixer as the only material which he has to actually handle is the cement.

The mixing outfit is Standard with the exception of the one-hoppered pug mixer which is only a modification of one type of continuous mixer. Tile manufacture has been handicapped up to the present time by lack of a mixer which is adapted to the special needs of tile business. An ordinary continuous mixer which, as a rule, proportions the materials mechanically, is in general use in the majority of plants simply because it mixes the materials in the pug mill or mixing trough so they can be used, but the proportioning is uncertain and inaccurate. One manufacturer states that he believes his mixer is costing him three or four dollars per day for the extra cement which he uses in an effort to avoid a lean mixture.

Although the batch type of mixer insures absolutely accurate proportions with but little if any extra labor, it is not adapted to mixing properly sand and cement. These materials, unless mixed with sufficient water to make them semi-fluid, tend to roll and ball rather than be rubbed and mixed together, as they are when coarse aggregate is in the drum or as they are in the pug. Most any of the standard batch mixers will mix thoroughly the dry sand and cement. The combination recommended in the layout, if properly operated, would overcome all the mixing troubles which have heretofore been experienced.

There are many things in connection with the operation of a tile plant which require constant attention, but only a few of the most important ones will be discussed here. One of these, and one which does not receive the consideration which its importance war-

rants, is the selection of materials. Only the best materials should be used because the most improved machinery and the best methods of manufacture cannot be expected to produce a good quality of cement tile from anything else. The question of cement may be dismissed with the suggestion that only Portland cement, which will meet the requirements of Standard Specifications,* be used.

Too often the consideration of cost plays the important part in the selection of the aggregate, but this is gradually becoming at least partially subservient to those qualities which will produce the

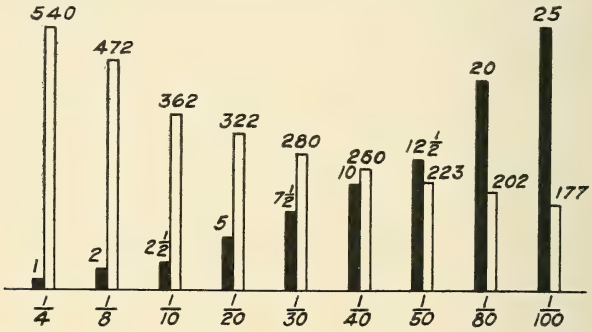

FIG. 4.—DIAGRAM OF RELATIVE AREAS (SOLID COLUMNS) FOR THE SAME VOLUME OF SPHERES OF DIFFERENT SIZES AND THE EFFECT OF SIZE OF SAND GRAINS ON THE STRENGTH (OPEN COLUMNS) OF THE MORTAR. THE LOWER FIGURES, ¼, ETC., GIVE THE DIAMETER OF THE GRAINS IN INCHES.

required results. Unless the aggregate is from a deposit of known quality, experience has shown that it should be tested. Heretofore there has been too much of a tendency to regard sand as sand. Natural deposits of sand or gravel may or may not be suitable for cement work and only an investigation of the physical characteristics will determine their relative value.

If only one sand is readily available tests may show that sand obtained from another source at an increased cost will improve the quality of the product sufficiently to justify the extra expense in

* See *Standard No.* 1.—ED.

obtaining it. Again, it is but rarely that any two sands are of the same quality and a study of their physical characteristics will determine which of a number, that may be used, is best suited for the work.

In the selection of sand too much emphasis cannot be placed upon the value of coarse and properly graded particles. It is a well-known fact that coarse sands are superior to fine sands for concrete, yet it is surprising how much fine sand is used for making tile. Fine sand has a greater number of particles per unit volume than coarse sand and consequently more points of contact between the grains, therefore, it requires more water for the same consistency, more cement to coat the extra surface area, and is more difficult to mix properly. The water required is sufficient to separate the particles by surface tension, increasing the voids in the mortar, thus decreasing its density and strength. The relative value of coarse and fine grained sands is clearly illustrated by Fig. 4. Assuming the short, solid column (1) to represent the surface area of a half bushel of spheres or sand grains, all of which are ¼ in. in diameter, the other solid columns represent the surface area of a half bushel of spheres or sand grains, all of which are of the diameter indicated by the figure under each column. The figures over the solid columns give the relative surface areas. This diagram shows very forcibly that with the same measured amount of sand the surface area varies greatly with the size of the sand grains. The surface area of a quantity of spheres or sand grains 1/100 in. in diameter is 25 times greater than the same quantity of spheres or sand grains ¼ in. in diameter. The open columns indicate the relative strength of test pieces made with one part cement and three parts sand separated into various sizes. As the various sized particles were separated from the same original sample it is reasonable to state that the difference in strength is entirely due to the size of the particles. It is interesting to note how rapidly the strength is decreased by the increase in the fineness of the sand.

A sand may contain a sufficient quantity of the smaller sizes to make it suitable for tile and still be coarse. This is illustrated by the results of tensile strength tests and granulometric analysis as shown in Figs. 5 and 6. The sands represented by curves 3 and 4 have an excessive amount of material retained on the No. 50 and

No. 80 sieves with a corresponding decrease in the strength of the mortar. Curves 1 and 2 represent good, coarse sand. Curve 1 shows too much sand of the coarser sizes to be entirely suitable for making the smaller sizes of tile, although it is an excellent sand for large tile and general concrete work. Curve 2 shows a coarse and excellently graded sand, and is ideal for making tile.

It is important that the individual sand grains be hard, as the mortar can be no stronger than the sand from which it is made. An aggregate, either sand or gravel, containing any consider-

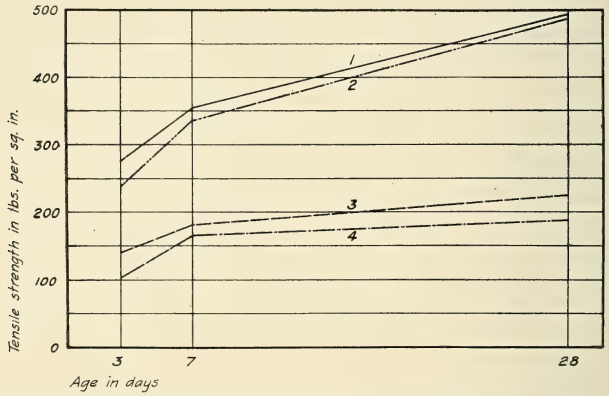

FIG. 5.—TENSILE STRENGTH OF 1 : 3 MORTARS USING SAND OF VARIOUS FINENESS AS SHOWN IN FIG. 6.

able quantity of shaley particles must be avoided as such material is weak and may be disintegrated by frost. In the glacial gravel deposits of the Upper Mississippi Valley these soft shaley pebbles are sometimes found in considerable quantities. In some of the river sands lignite proves troublesome. Lignite is a poor quality of coal containing the texture of the wood from which it was formed, black in color, brittle and easily broken between the fingers. The larger sizes may be removed by screening, but even so, the smaller particles remaining expand during the hardening of the mortar and those near the surface "pop out," disfiguring and weakening the tile.

Sand should be free from clay, loam, mica, or an excess of fine material, even though siliceous. It is not the presence of the clay or loam to a limited extent, but the manner in which it is present that does the damage. The sand grains themselves must be free from a coating of foreign material. A simple and effective test for detecting such a coating on the sand grains is to wash a small sample by shaking it in a bottle or jar with water, and after pouring off the water and drying the sample, to compare it with some of the untreated sand. If there is any appreciable difference in the color of the two there is evidently a coating on the grains. Such sand will produce a weak mortar because the cement cannot

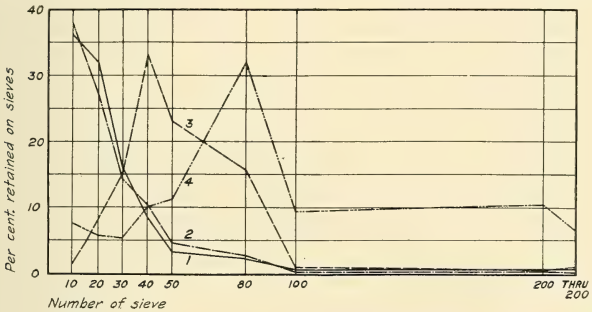

FIG. 6.—GRANULOMETRIC ANALYSIS OF SANDS USED IN MORTARS WHOSE TENSILE STRENGTH IS SHOWN IN FIG. 5.

form a good bond with the aggregate, and the fracture of the mortar shows that the grains pull out of the cement, leaving little pockets lined with a thin film of clay. The use of such sand is apt to result in failures.

When cement tile were first made by machine very lean mixtures were used, but it was soon learned that this was objectionable, and the best practice now is to use for the tile 10 ins. and smaller, a mortar composed of one part Portland cement and not to exceed three parts of fine aggregate. For the larger sizes it is more economical to use concrete instead of mortar, the diameter of coarse aggregate being limited so as not to exceed one-half the thickness of the tile walls. The proportions for the concrete should

be one part cement to not more than two and one-half parts sand, with as much coarser aggregate, up to four parts, as can be used. With some materials the mixtures may not carry four parts of coarse aggregate without causing trouble by the green tile falling down when stripped.

One of the most important things to watch in making tile is the consistency of the mix. The amount of water which it is possible to use in the mixture for machine-made tile is at the best not sufficient to develop the full strength of the cement, and for this reason the mix should be made as wet as can be used. When the jackets are removed the tile should show water marks on the outer surface, that is, web-like markings indicating free moisture. Although it is generally recognized among cement tile manufacturers that tile showing water marks are the strongest, this matter is not given sufficient attention. The tile made from a dry mix are not so apt to stick in the jackets as those made from a wet mix and they stand up better, which probably explains why so many manufacturers use a dry mixture. Sticking to the jackets is easily overcome by holding the tile, before removal of the jacket, horizontally and hitting the end against the table of the machine just enough to loosen the tile from the jacket. The same thing is accomplished by setting the tile down hard on the car, but this thickens the walls at the end and is objectionable.

Because of an insufficient amount of water in the original mixture, every particle must be conserved, which is impossible if the tile are cured under normal atmospheric conditions. The object therefore, in steam curing is to create an atmosphere saturated with moisture in a chamber sufficiently air tight to eliminate all possible draughts. It is not the intention to supply moisture to the tile but to protect it perfectly against the loss of any moisture which was introduced when mixing the concrete. As a saturated atmosphere is required, the use of dry steam or steam under pressure is out of the question. Some plants are operated to-day by the use of such steam and in parts of the curing chamber fairly good results are obtained because the steam condensed in these parts while the product placed near the point at which the steam enters the curing chamber is poor owing to the drying action of the steam. This condition is naturally aggravated in warm weather because the steam does not condense so readily. Only by the use

of steam curing is it possible to obtain uniform conditions throughout the year.

The safest and most satisfactory way of admitting steam into the curing chamber is through a perforated pipe which is placed, perforations down, in a water-filled trench in the concrete floor of the room; the steam passing out into the atmosphere through the water naturally becomes saturated. The floor in the curing chambers is sloped toward the trench so that the water in the trough is constantly replenished by condensation. In this way there is little danger of excessive heat, for as long as the steam passes through water the temperature, no matter what it is, cannot have a drying effect. The nearer the temperature approaches the maximum possible with this method, the more efficient does the curing chamber become, for moist heat accelerates the hardening of the tile. With this scheme exhaust steam will not furnish sufficient heat during cold weather, therefore it is best to take live steam direct from the boiler. Tile should be run into the curing chambers as soon as made, but it is not advisable to start the steam until the tile have stood about an hour, especially in cold weather, because condensation will be very great until the chamber and the tile become heated up, and the gathering of moisture on the tile may be sufficient to damage them.

To operate the suggested plant to full capacity on all sizes of tile will require a foreman and about twenty men distributed as follows: one engineer, one man running the mixer, one man feeding each tile machine and one operator on each machine, one man removing jackets from small tile, one handling cars and four yardmen who remove the tile to the yard, load wagons and do odd jobs about the plant when not otherwise engaged. Two men will always be needed to wheel the large tile from the machine and sometimes three, one man removes the jackets and two men, with the help of the one man sometimes used for wheeling tile can remove the tile to the yard, load wagons and cars; a night man will be required to keep up steam, clean, repair and change the machines.

In order that no tile may be removed from the curing chambers too soon it is well to have the man handling the cars of small tile to the curing chambers keep a record of the number of tile of each size on each car and the number of cars placed

in each chamber. The man removing the jackets from the large tile can keep a record of the number of each size placed in the different chambers; these records to be handed in at the office twice a day. By this practice a memorandum showing the number of cars and the number and size of large tile in the different rooms which are to be put in the yard that day, may be given to the yardmen every morning. From these records can be compiled the daily output and the stock on hand. Some allowance must be made for breakage, otherwise the number of tile made in the yard will not agree with the number of tile made by the machines.

The arrangement of the stock piles should be such that the wagons may be loaded with any size and leave the yard by crossing the fewest number of tracks. This is accomplished best by piling the tile in long rows parallel to the track. The distance between tracks may be anything desired. Some prefer not to have a greater distance than 20 ft. between the center line of the track and the center of the roadway. Others have found a distance of 50 to 75 ft. to be a convenient width. In the latter case, to avoid carrying the tile long distances when unloading the cars, they are rolled down a chute about 14 ins. wide with 4 in. sides and 20 to 24 ft. long, one end of which is placed on the edge of the second deck of the car. The tile can be rolled 30 to 35 ft. and this method proves very satisfactory for all the small tile larger than 5 ins. If the sales are large during the winter months, it is desirable to place the stock piles on a layer of straw or other material, as it is very often that the lower row of tile freezes to the ground which prevents the wagon from getting close to the pile, and they cannot be loaded without much extra labor, if something of this kind is not done.

A car load of 30,000 lbs. will contain approximately:

4300	ft. of	4	in. tile	300	ft. of	18	in. tile
3400	"	5	"	250	"	20	"
2750	"	6	"	210	"	22	"
2000	"	7	"	200	"	24	"
1500	"	8	"	175	"	26	"
1100	"	10	"	150	"	28	"
800	"	12	"	125	"	30	"
600	"	14	"	110	"	32	"
500	"	15	"	85	"	36	"
400	"	16	"				

It is the best practice in loading tile either in wagons or cars to lay them horizontally instead of standing them on end. They should be packed as closely as possible with no play; especially is this true in loading cars. Good cement tile when properly loaded can be hauled or shipped considerable distance with little or no breakage. Switching partially loaded cars will cause excessive breakage.

DISCUSSION.

Mr. Losse. MR. LOUIS H. LOSSE.—I would like to ask the speaker how much money is represented in one of the model layouts.

Mr. Powell. MR. C. M. POWELL.—I have estimated the building to cost practically $9,500 and the equipment about the same, which would make the completed plant all ready to start, cost $19,000. This covers a plant to manufacture both small and large tile.

The cost of a plant for making small tile only would depend very much upon the amount of equipment, but I might say that a small tile machine would cost from $1,250 to $1,500, a mixer of the type described will cost at least $350, and the cars about $19.00 a piece. The boiler and engine will cost about whatever you want to pay for them, as some plants have been able to use second-hand boilers very satisfactorily.

Mr. Spackman. MR. HENRY S. SPACKMAN.—Referring to the table on the effect of various sized grains of sand with respect to cement, at what period were those tests made, or do they represent an average extending over quite a period?

Mr. Powell. MR. POWELL.—I understand they were short time tensile tests, the age of test pieces being 60 days.

ADDITIONAL NOTES ON STEAM CURING PLANTS.

By F. S. Phipps.

There has been no factor in the cement industry that has been of more benefit to the manufacturer of cement products than the steam curing rooms. It has proven to be the only method of curing blocks and ornamental work in the winter time when the weather is exceedingly cold, and has proven a great benefit to the manufacturer in exceedingly hot weather. There have been advocated three or four classes of curing rooms, but only two classes have proven successful. Among the new developments may be named the construction of curing rooms, the handling of the steam, the adding of water to the curing rooms and the construction of a hood to cure large pieces of cement work where it has been impossible to convey same into the curing rooms.

The construction of the most successful curing rooms has been of blocks and concrete arches or entirely of reinforced concrete, whichever is most convenient to the builder. The arch seems to give the most satisfactory curing room, the steam passing up the side walls over the arch and down on the blocks. Any moisture gathered on the arch will run down the sidewalls instead of dripping off on the blocks as with a flat ceiling. Another successful curing room is one built of galvanized sheet iron which is nailed upon timbers or side walls of the building, one side of the galvanized iron having a slot about ½ in. in length where it is nailed to the wall, allowing free expansion and contraction of the iron, so as not to draw the nails from the wood. There have been several mistakes made by nailing the sheet iron directly on the timber, and when the heat and cold contracted and expanded the curing rooms, the nails gave way, producing cracks whereby steam could escape.

All curing rooms should have a ventilator at the front to allow the steam to escape through the roof of the building. The most successful door for curing rooms has been made of heavy canvas, which allows the air from the outside to condense the steam in the rooms, giving the blocks more moisture with less steam than in the case of a wooden or iron door.

The curing hood referred to is made of sheet iron and shaped like a funnel or made square or oblong as desired. This hood is used where large pieces of stone are manufactured throughout the building and are required on the job before the regular time for curing. It is placed over the cement block and steam admitted through a hose. On the top of the hood there should be a pipe, not more than 3 or 4 in. in diameter, which is used as a ventilator. It is well to have an opening on one side of the hood and cover it with canvas which will act as a condenser upon the steam. This will allow the curing and removal of large slabs or sills within 3 or 4 days after making, whereas otherwise 10 days to 2 weeks are required. It is very useful when large slabs and sills cannot be transferred to the curing rooms. With this hood, a steam pipe extending through the factory and a piece of hose, these slabs or sills can be cured and sent out much sooner than under the old method of curing. There have been cases where the curing hood has been used on outside work, where a mixer is used with a steam engine. The hood has been taken on the job and large slabs of walks have been cured with it when the weather unexpectedly turned exceedingly cold. If the walk should be 40 or 50 ft. long the hood is moved up and down the walk to keep it warm. With the additional expense of two men and a little coal, the contractor saved the expense of replacing the walk as well as a reputation for bad work, although it really would not have been his fault if the walk had been spoiled on account of the weather.

There are three or four methods for handling the steam in the curing rooms. One is using exhaust steam from an engine which in a great many cases has proven satisfactory; another, using the steam from a boiler carrying 50 to 60 lbs. of steam and reduced down to 2½ lbs. Another method to be recommended is low pressure steam or wet steam, direct from a boiler carrying the water gage almost full of water; this does not heat up the curing rooms as the other process. The curing rooms can receive a large or small amount of steam, regulated just as the work demands.

Another way of handling steam is the installation of tanks of water along the side of the curing rooms and turning steam into them to boil the water or running steam pipes through these tanks, causing the water to boil and create a steam in the curing

rooms. Sprayers or sprinklers have been used to some advantage, especially upon poured or wet work, which causes the material to bleach out, the work being placed in the curing rooms some 12 to 20 hours after being made, when it is hard enough to receive sprinkling and then steaming, which hastens the curing of the product and bleaches it out very much whiter than in any other manner.

The following incident will fully illustrate the advantage of steam curing rooms during hot weather. We received three cars of cement, which at first trial, under the system of curing in use, proved to make a very poor product, in fact we were not able to use several cars of blocks for the reason the cement seemed to set before it received proper moisture. We came to the conclusion that it was poor cement, but upon testing same we found it to be exceedingly strong. The tensile strength of briquettes 24 hours in moist air was 435 lbs. per sq. in; 6 days in water and 24 hours in moist air, 830 lbs., and 27 days in water and 24 hours in air, 1,008 lbs. The cement was very quick setting—smaller batches were used, placing the blocks in the curing rooms and giving them steam in less than one-fourth of the usual time, and three cars of as good, if not better, product were turned out than during any part of the season. Without the steam process it would have been impossible to use that cement for blocks or ornamental work. Of course, it could have been used in poured work and did work very successfully in that way, but the disadvantage of having to return those cars and get other cement would have cost a great deal more than the construction of the curing rooms. The writer is of the opinion that every manufacturer of cement products who wishes to run his material during the entire year and to manufacture it in the best possible way, both for himself and his customer, is working to a great disadvantage by not having steam curing rooms to take care of manufactured product. The expense is not very great and the benefits are many. The writer is not willing to estimate the cost, as the material used in constructing and equipping curing rooms will vary in different parts of the country and depends on the amount of space it is desired to cover with curing rooms.

Mr. Watson. Mr. Charles D. Watson.—One of the subjects of most vital importance to the proper development of cement products, is the question of costs. It is the competition of the man who does not know what his product is costing, that is doing so much to retard the universal manufacture of high class material. Any man who has the sand thinks he can make concrete blocks and be successful. Outside of the location and arrangement of a plant, the greatest mistake is made in improper cost keeping.

As an illustration I would mention a recent competition on concrete stone for a building of considerable size, in which the seven bids submitted ranged from $22,000 to $62,000. On investigation it was found that the lowest bidder had never done concrete work and had never made concrete building stone. The man stated he had experience in brickwork and knew all about laying concrete sidewalks as well as the cost thereof. The others interested in the firm had no experience and relied absolutely on this man.

It is a necessity to keep careful costs in order to know the actual cost of the product. The shop management and overhead expenses are rarely figured and few realize their amount.

Mr. Pierson. Mr. G. N. Pierson.—The competition I fear more than any other is that of the man who does not know what a block costs him and does not know when he is selling below cost.

Two or three months ago I took up the question of costs in my factory and found that the variation in the general expenses, in the improvement account and in several accounts which I had to carry, made a varying cost on different blocks. Now I have created a sinking fund, you might call it, for all overhead expenses. That is, I averaged the general and incidental expenses for the last five years, interest, taxes and improvements, and charge off each month a set amount which is a little larger than the average for the last five years. This gives a uniform overhead charge, an invariable upon which to figure the cost of blocks.

The greatest difficulty has been the variations in the sand.

When the sand is damp it does not go through the continuous mixer as rapidly as when it is dry. I would like to know how to overcome this. Mr. Pierson.

On the Volume of Wet and Dry Sand.

Mr. C. W. Boynton.—No one present, I presume, would undertake to tell how to overcome the difficulty met with in the use of a mechanical proportioner. It will be hard to overcome the trouble complained of and, at the same time, have the machine practicable. I do not believe that the average mechanical proportioner is a fit thing for a cement products plant. In using these machines one never knows just what the proportions are. The fineness of the sand and the moisture contained in it, as well as the humidity of the atmosphere affect the working of the machine. I believe the "Pug Mill" type of mixer, which is commonly used with a mechanical proportioner, usually gives very good results in mixing sand and cement. The material, however, should be proportioned accurately by the use of measuring boxes, chutes or some such device, for it is only in this way that one will know positively that the proportions are correct. In my opinion the best way to handle the materials is to measure them accurately and then pass them dry through a drum or some other type of batch mixer from which they may enter a pug mill, where the water is added. By rebuilding these mixers and using the whole length of the pug as a wet mixer, very satisfactory results will be obtained. Mr. Boynton.

In studying the manufacture of cement drain tile, I have had an opportunity to carefully note the behavior of mechanical proportioners, and in every case I have found them unreliable. For instance, if a mixture of 1:3 was aimed at, the machines have varied all the way from 1:2 to 1:5. Unfortunately the manufacturer does not know when this variation occurs, for it is usually not discovered until the machine is again checked. This necessarily means that some of the product is of inferior quality, while other contains more cement than is necessary. A plant cannot be economically operated unless it is at all times under the control of the operator.

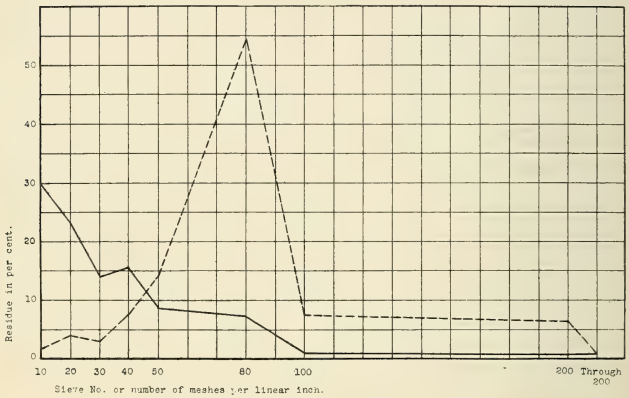

FIG. 1.—GRANULOMETRIC ANALYSIS OF SAND.

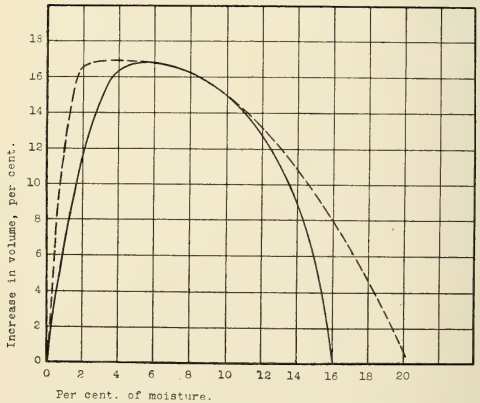

FIG. 2.—DIAGRAM SHOWING INCREASE IN VOLUME OF SAND DUE TO VARIOUS PERCENTAGES OF MOISTURE.

Mr. Pierson. Mr. Pierson.—It would be necessary to have some system for drying the sand, because measured in any given volume the variations in the dampness would make a variation in the actual quantity of sand obtained.

Mr. Boynton. Mr. Boynton.—It is a fact that the moisture contained in sand will affect its volume regardless of the means adopted for measuring it, and, where possible, a practically dry sand should be used. When dried sand is not used, the moisture should be determined and the volume corrected accordingly.

To obtain information as to the effect of moisture on the volume of sand and to check the results of others, I experimented

Table I.

Per cent. Moisture.	Per cent. Increase.	Volume in cu. ft. Necessary to Make a Mixture of				
		1 : 2	1 : 2.5	1 : 3	1 : 3.5	1 : 4
0	0.0	2.0	2.5	3.0	3.5	4.0
1	6.5	2.13	2.66	3.2	3.73	4.26
2	11.0	2.22	2.78	3.33	3.88	4.44
3	14.5	2.29	2.87	3.43	4.00	4.58
4	16.5	2.33	2.92	3.50	4.08	4.67
5	16.8	2.37	2.92	3.50	4.08	4.67
6	16.8	2.37	2.92	3.50	4.08	4.67
7	16.6	2.33	2.92	3.50	4.08	4.67
8	16.2	2.32	2.91	3.49	4.07	4.65
9	15.8	2.31	2.89	3.47	4.05	4.63
10	15.0	2.30	2.87	3.45	4.03	4.60
11	14.2	2.28	2.86	3.43	4.00	4.58
12	13.0	2.26	2.83	3.39	3.95	4.52
13	11.3	2.23	2.79	3.35	3.91	4.47
14	9.0	2.18	2.62	3.27	3.82	4.36
15	6.0	2.12	2.65	3.18	3.71	4.24
16	0.0	2.00	2.50	3.00	3.50	4.00

with two sands, one a fine plastering sand and the other a Torpedo sand. The granulometric analyses of these sands are shown graphically in Fig. 1. The full line represents the Torpedo sand and the dotted line the plastering sand.

In determining the effect of moisture on this sand a metal vessel of one cubic foot volume was used. The results are shown in Fig. 2. It will be noticed that the volume of the coarse sand is slightly less affected by moisture than the fine sand. The results agree very well with those obtained by others.

It is evident that to obtain uniform proportions when using sand containing various percentages of moisture, a correction should be made as indicated in Table I. From a study of this table it is

Mr. Boynton. clear that accurate proportions and cost can only be obtained by adjusting the volume of the measuring box to agree with the change in the moisture in the sand. I should think that under ordinary circumstances the moisture in sand should be determined at least once a day.

A convenient and simple way of determining moisture is to use an ordinary scoop or platform scale accurately balanced and sensitive to ounces. Weigh 12½ lbs. of the sand, dry to a constant weight, re-weigh and divide the loss in weight, expressed in ounces, by 2, which will give direct the per cent. of moisture in the sand.

To make the data available for reference I wish to append to my discussion extracts from various authorities on the effect of moisture on the volume of sand.

EXTRACTS FROM *Concrete, Plain and Reinforced*, By Taylor and Thompson.

Effect of Moisture on Sand and Screenings.—Moist sand occupies more space and weighs less per cubic foot than dry sand. * * * The explanation of this apparent anomaly lies in the fact that a film of water coats each particle of sand and separates it by surface tension from the grains surrounding it. * * * Fine sand, having a larger number of grains, and consequently more surface area, is more increased in bulk by the addition of water than coarse sand. The volume of coarse, broken stone and gravel is but slightly, if at all, changed by moisture, while small broken stone, composed largely of particles of less than ½ inch in diameter is affected like sand.

If a small quantity of water is poured into a vessel containing dry sand, the bulk is not increased because of the inertia of the particles, but if the sand after moistening is dumped out and then turned back into the vessel with a shovel or trowel, its bulk will be increased. On the same principle, a sand bank does not swell in bulk during a shower, but the effect of the moisture is shown in the excavated material as soon as it is loosened with the shovel, and therefore its loose measurement for concrete or mortar is effected.

The diagram in Fig. 66, plotted by Mr. Fuller* from experiments upon a single sample of natural sand mixed by weight with varying percentages of water, illustrates the effects of moisture upon the actual percentages of voids in sands loose and tamped. The volumes produced by varying degrees of compacting are located betwen two curves. It is noticeable that both the loose and tamped sand increase in volume with the addition of water and reach a minimum with about 6 per cent.

* *Engineering News*, July 31, 1902, p. 81.

of water, then decrease, and finally, when saturated, return to slightly less than their original dry bulk. The same sand, it is seen, may contain from 27 to 44 per cent. of absolute voids, according to the percentage of water and the degree of compacting. Mr. Boynton.

The percentage of water by weight which will give the greatest bulk,—corresponding, of course, to the largest percentage of absolute voids,—varies with different sands from 5 to 8 per cent.

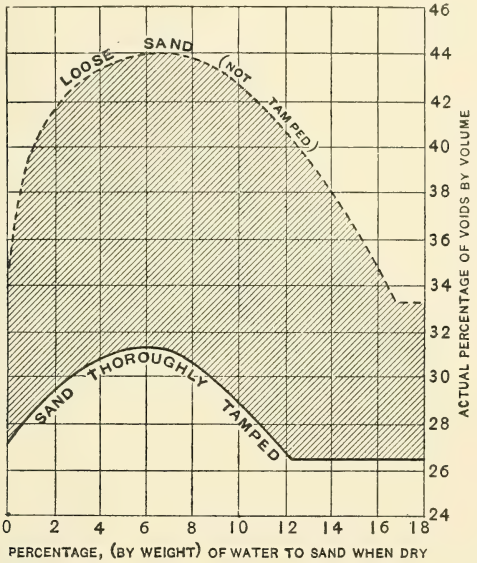

FIG. 66.—PERCENTAGE OF ABSOLUTE VOIDS IN A NATURAL BANK SAND CONTAINING VARYING PERCENTAGES OF MOISTURE.

The actual variation on different days in the percentage of moisture in a natural bank sand was found by the authors, in a series of experiments, to range from 1½ to 5¼ per cent. of the total weight or from 2½ to 7¼ per cent. of the bulk of the moist sand.

The maker of concrete is especially interested in the influence of moisture upon the bulk of sand and upon its voids (1) because of its effect upon the actual measurement of sand used in construction work, and (2) because of its effect upon his experimental determinations of proportions.

Mr. Boynton.

Rather incomplete experiments of the authors tend to show that the actual effect of moisture upon the volume of sand used in concrete and mortar may often be less than would naturally be inferred from the various experiments cited, and depends largely upon the processes of handling the sand. For example, fairly dry sand (3 per cent. moisture) shoveled by laborers from the pile into the regular sand-measuring box weighed 454 lbs., while after a rain, the sand (with 5 per cent. moisture) shoveled from the pile into the same box weighed 464 lbs., that is, the moist sand was slightly heavier than the dry. Further handling reversed these relations, for on weighing these two sands in a half cubic foot measure, the moist sand, as we should expect, was lighter than the dry.

The explanation of this apparent discrepancy is undoubtedly due to the fact that as the rain which affected the moisture occurred after the sand had been excavated and piled near the mixing platform, its bulk, as suggested, was not affected. The laborers handling the moist sand took large shovelfuls and the arrangement of the grains was not greatly disturbed. If the sand had been excavated after the rain, the handling with shovels and dumping from the cart probably would have re-arranged the grains so that the moist sand would have weighed less than the dry in the large measure as well as in the small box.

Mr. Feret* calls attention to the fact that mortars of nominally the same proportions are richer in winter than in summer because of the greater amount of moisture in the sand, which, by increasing its bulk, reduces the absolute volume of the grains in a unit of measure. On the other hand, the mortars are leaner in dry than in damp weather because the sand has greater density when dry.

EXTRACT FROM *Cement and Concrete*, By L. C. Sabin.

Effect of Moisture.—The effect of a small amount of moisture on the bulk of a given weight of sand is not usually appreciated, but it may easily be shown that it is very marked. * * * It may be noticed that the difference in bulk between moist and dry sand is greater when the measurements are made "loose." * * * This subject is of great importance in proportioning mortars because, in construction, the amounts of cement and sand are usually measured. Suppose it is desired to use a mixture of 100 lbs. of cement to 400 lbs. of sand, and for convenience we will suppose the packed cement and dry sand each weigh 100 lbs. per cu. ft. If now we use damp sand, containing about 3.5 per cent. water, instead of dry sand, and measure the materials, we would have 4 cu. ft. of damp sand to 1 cu. ft. of cement; but damp sand would contain only about $4 \times 75 = 300$ lbs. of dry sand, and we would really have a 1:3 mixture instead of 1:4.

* *Annales des Ponts et Chaussees*, 1892, II, p. 26.

The Effect of Moisture on the Bulk of Dry Sand. Mr. Boynton.

Engineering News, Vol. 60, No. 8, p. 211. Aug. 20, 1908.

On page 176, Taylor and Thompson's *Treatise on Concrete*, attention is called to the fact that dry sand upon moistening occupies more space than when dry. The explanation given is that a film of water coats each grain of sand, separating it by surface tension from the grains surrounding it. A little farther on reference is made to the fact that if dry sand is completely saturated with water it will occupy less space than when dry. It seems to the writer that this is a contradiction, as in both cases, the sand grains are covered with a film of water, and were it the film of water around each grain that caused the increase in bulk, the results obtained would be the same whether the sand was merely moistened or completely saturated.

The explanation the writer has to offer for the increased bulk in dry sand upon moistening is as follows: When dry sand has only sufficient moisture added to it to make it of what might be termed a "sticky consistency" and is then disturbed so as to permit of rearrangement of the grains, the innumerable number of fine particles which previously rested in the voids of the coarser particles adhere or stick to the larger particles, and in this manner prevent the sand from packing as closely as when dry, it being clear that when perfectly dry the sand is what might be termed "slippery," there being no tendency for the finer particles to coat the larger, thus permitting the large particles to pack closely together and the fine ones to occupy the voids between. This seems to be proved by the fact that upon addition of sufficient water to produce saturation, the sand again returns to the same or less bulk than when dry, the water in this case acting as a lubricant and permitting the same close packing as when perfectly dry.

Harold D. Elfreth, 1700 N. 12th St., Philadelphia, Pa.

The Effect of Moisture on the Bulk of Dry Sand.

Engineering News, Vol. 60, No. 12, Page 310.

That moist sand occupies more space than the same amount does when dry is a fact well known to most engineers. But that this fact might have any bearing on practical work is seldom considered.

Some time ago while gathering cost data, I noticed what at first glance seemed to be an error in observation. In mixing a rather large amount of Portland cement mortar the following quantities were used: 10.3 cu. ft. of sand, 330 lbs. or 3.2 cu. ft. of Portland cement and enough water to make a thin mortar. The quantity of mortar obtained was only 9.4 cu. ft. The cement was weighed on a Fairbanks scales and the sand was measured in specially constructed measuring boxes,

Mr. Boynton. which had previously been checked by both the engineer and the contractor's representative.

The sand used was bank sand obtained near Coshocton, Ohio, being locally known as Coshocton sand, a considerable quantity being larger than ¼ in. The sand as delivered on the works contained about 5 per cent. of moisture by weight.

Experiments in regard to the increase in volume of sand for various

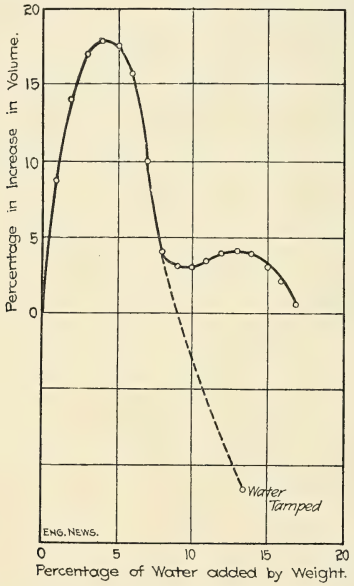

DIAGRAM SHOWING EFFECT OF MOISTURE ON BULK OF DRY SAND.

percentages of moisture gave the curve shown in the accompanying cut. The sand was measured loose and the percentages of water to dry sand were taken by weight. It should be noted in this curve that when the percentage of water added to the sand became more than 8, considerable difficulty was experienced in obtaining an even mixture. A great number of minute air bubbles were formed which would very seriously affect the results. The dotted line would, in the writer's opinion, be more correct.

To determine if the gain in volume on account of moisture was

excessively large for Coshocton sand, experiments were made with various other sands. The results indicate that the finer the sand and the more uniform the grains, the greater will be the increase in volume. Mr. Boynton.

A. Stellhorn, 1231 S. High St., Columbus, Ohio.

Mr. F. S. Phipps.—I have been trying for eleven years to get an accurate cost of sands, but have found it a very hard proposition. We come the nearest, I think, at the present time. The sand is purchased in carload lots and it is shipped about 60 miles, consequently most of the water has run out of the sand, and it is fairly dry. Of course, it is not dried, but in measuring it by volume gives very accurate proportions. I have tried the continuous mixers, the batch mixers and have pumped sand direct out of the river, using it that day, and have shipped it short and long distances. And the way the cost of our sand is now obtained is by paying for it by weight. The car is measured and allowance made for settling, jolting down of the sand, and then a measure is used on all work. For a mixture of 1 : 4 a 4 cu. ft. measure is used. If a car comes through very quick and the sand used right away because the supply is exhausted, just a little more sand is placed in the box than if it were perfectly dry. The proportions are very much better as well as the quality of the produet, than I ever could make with a continuous mixer. Mr. Phipps.

I have found the same trouble Mr. Boynton spoke of when using continuous mixers and never was sure of the proportions. If the sand was continuously dry, as in the hot summer months, and especially when it takes a week or ten days to get a car through, the sand would go through the mixer quickly and the product was very poor. And my experience has been all through that the batch mixer is much more accurate than any continuous mixer I have ever seen work.

On Steam Curing.

Mr. Rudolph J. Wig.—Some rather extensive investigations on steam curing revealed several variables. One important factor is the age at which the product is placed in the steam curing chamber and this seriously affected the results. Better results were obtained by allowing the concrete to get an initial set before it was placed in the steam curing chamber. This is not the prac- Mr. Wig.

Mr. Wig. tice now, but concrete treated in this manner showed a higher strength in the testing machine. Another result was that the longer the period of steam curing at atmospheric temperature the greater the strength. That is, the strength was directly proportional to the time of curing. In the first few hours up to 12 hours practically nothing was gained; in fact, curing the product only 12 hours and then removing it from the chamber really injured rather than helped it. It seems that if heated through and then taken from the chamber the product is dried out, the cement does not harden properly and obtains very little strength. We tried steaming for 17 hours, leaving the product in just over night, with better results, but 48 hours or even 36 hours gave very good results. The best results were obtained by allowing the product to attain its initial set first, then placing it in the chamber and leaving it for at least 36 hours. Some were steamed as long as 7 days.

The test pieces were of two kinds, cylinders 8 in. in diameter by 16 in. in length and $12 \times 12 \times 24$ in. concrete blocks. Limestone concrete and well graded river sand mortar mixtures were tested, of dry, wet and sloppy consistencies. We had no success in steam curing very wet or sloppy mixtures.

Mr. Boynton. MR. C. W. BOYNTON.—It seems to me that a product like cement tile, which of necessity has very thin walls, might be damaged by allowing it to stand exposed to weather conditions for any length of time before it is placed in the curing chamber. Cement tile, usually made of comparatively dry mixtures, should be protected against loss of any moisture, for under the most favorable condition the tile mixture does not contain as much moisture as is required to develop the full strength of the cement.

Mr. Wig. MR. WIG.—I think Mr. Boynton's point is well taken. Some tests on thin sections showed such results. The thin sections did not act the same as the large concrete blocks or larger specimens; and it is a matter that needs a great deal more investigation and much more thought given to it.

I do not think it right to assume that all steam-cured blocks are the same. There is a great difference in the manner of steam curing. Temperature also plays a considerable rôle. That is, you can steam cure. A block can be cured by steam so as to

impair its strength and also so as to strengthen it considerably. We made a series of tests with varying temperatures of steam, that is, using pressures running up very high with varying temperature and found there was a decided difference in its effect depending upon the quality of the steam and its temperature. That is, with dry steam, raising the temperature without having condensation on the block you really injure the block; raising the temperature and getting condensation on the block you increase the strength of the block; or using high pressure and high temperatures and passing the steam through a water body you get a thoroughly saturated steam, and increase the strength remarkably. We made some tests with pressures as high as 80 lbs. per sq. in. and got some very remarkable strengths. These tests were made on concrete blocks, large sized specimens, and a strength of 4,000 to 5,000 lbs. per sq. in. was obtained in a 1:4 concrete, testing when only 2 days old having been steamed for 17 hours. Using a 1:8 concrete gave practically the same results proportionately. Mr. Wig.

Mr. H. S. Spackman.—I think Mr. Boynton raises a point that is well taken, as in winter it might be advisable to take the block from the steam chamber at different times. I have in mind an experience with sand lime brick—it is not quite the same case—but I think weather conditions would act the same in connection with the cement brick plant. The owner of the plant noticed while they were using exactly the same materials and methods that the brick made in the winter showed much less strength than in summer. The question was submitted to me, as the manufacturer thought possibly taking the hot brick from the cylinder and storing out of doors in freezing temperature might affect them. We carried out a series of tests. In the summer the bricks were taken directly from the cylinder and placed into an ordinary ice cream freezer packed with ice and kept them there 24 hours. When tested they showed exactly the same relative loss of strength that the bricks in winter did. I do not know what would be the effect of taking green concrete material from a steam chamber and putting it out of doors, but I think it would be the same. Mr. Spackman.

Another point, I think it is advisable to allow the cement to set before it goes into the steam, and if any such setting is to

Mr. Spackman. be allowed, it should be done in a damp room without increase in temperature.

ON EXPANSION AND CONTRACTION.

Mr. Gill. MR. J. R. GILL.—I am bothered a good deal with cracks developing in concrete block walls where the blocks were used green, and would like some information on that line. Does the concrete contract as it hardens, and if so, what is the actual amount of contraction for any given period of the hardening process, and does this contraction take place more rapidly during the earlier hardening stages or later; and what is a safe rule for the use of concrete blocks as to their age in order that the cracks due to hardening shall not develop in the wall?

The amount of contraction if known would determine the lengths of wall in which a noticeable crack would be likely to occur. Is there a shrinkage that takes place due to the hardening process, independent of the change of temperature?

Mr. Wig. MR. R. J. WIG.—Thompson and Taylor made some investigations on that subject and found that concrete contracted very slightly in air and expanded somewhat in water, but this expansion is exceedingly slight. It is a question of moisture somewhat and of the quality of the concrete or of its exposure. But this expansion or contraction is so very slight that I do not think it has been measured for concrete. I should not think this would make any difference in a concrete block wall in so far as causing cracks in the wall.

Mr. Gill. MR. GILL.—The specifications for concrete blocks require them to be used in not less than three weeks, and at the time those specifications were adopted, this period was set for the reason that there had been trouble with the cracking of walls apparently due to this cause. The crack either follows the joint or goes through the block and does not develop until some time after wall has been built, sometimes a month.

The President. PRESIDENT HUMPHREY.—I think the difficulty is not due to the contraction of the cement in the process of hardening, but due to greenness and lack of strength to resist the transverse pressure of the wall. If the strength is not good there will be strains developing and cracks develop later.

MR. GILL.—The cracks are the same size at the bottom and the top and are about 1/16 in. wide from top to bottom in long stretches of wall, and they usually do not occur in short stretches. I have had that experience very frequently. There will develop two or three cracks in the length of 100 ft. Mr. Gill.

PRESIDENT HUMPHREY.—It is possible if the crack is the same size that it is due to the contraction of the wall. A shrinkage crack due to hardening would not produce a crack of that kind. The President.

MR. GILL.—I have noticed in foundations the same tendency where there was no loading to amount to anything. I know of one particular plant that made a practice of using the blocks quite green, ten days or two weeks old, and almost all their foundations, twelve or fifteen different structures, developed those cracks. In almost every case they were the same size top and bottom,—1/16 in. They appeared in shorter walls where they used blocks very green. Mr. Gill.

MR. WIG.—Let us take as an illustration a monolithic construction 100 ft. in length. If it were due to contraction in the cement and not caused by temperature change, cracks would develop much more rapidly and to a greater extent in the monolithic construction than in a concrete brick wall. If it were due to the contraction of the cement the cracks would be just in the individual block. And if this were true of a very large monolithic construction of 100 ft. or more, cracks would develop all through the concrete. I should think that would show it was impossible for the concrete to contract sufficiently to cause cracks in a small block, that is, a block 20 or 30 in. long. The wall is not a monolith after it is laid up of concrete blocks; it has joints and these cracks are apparently due to the expansion and contraction of the whole wall caused by atmospheric temperature changes, the wall expanding with the heat of the day and as the temperature lowers endeavoring to contract, but unless the individual blocks are strong enough to resist the tensile stresses put upon them at this time, causing cracks to occur between the blocks, the individual blocks may be cracked. Mr. Wig.

MR. F. OLDE.—I think the cracks were undoubtedly caused by a contraction and expansion due to atmospheric conditions. I Mr. Olde.

Mr. Olde. have had the same experience in walls of considerable length and the cracks invariably appeared at the weakest part of the wall, such as over a lintel or some place of that kind, where the resistance to contraction strain was weakest. We had some experience with foundation work that was put up during the winter, and that would develop the same cracks, right straight up and down, at the weakest part of the wall. The cracks would develop immediately after in the spring, after the temperature changed. Uneven settlement was not the cause as the cracks appeared so evenly. They would appear in the wall where there was no compressive strain.

In order to avoid cracks it is best to provide continuous vertical joints in the wall at distances of about 25 to 40 ft. This has been done in a considerable wall length and no trouble developed.

Mr. Chandler. MR. DANA G. CHANDLER.—I had the same trouble before curing with steam. My factory building is 80 ft. long and built of water-cured blocks with 70 ft. addition of steam-cured blocks and haven't a crack in the whole building. It seems that the curing has something to do with this cracking.

Mr. Stimpson. MR. H. T. STIMPSON.—A long brick wall has no cracks. A brick wall is composed of small units which are previously manufactured and assembled in very much the same way as cement blocks. It would seem that there is trouble in the cement block. On the other hand, a monolithic wall about 250 ft. long, built about seven years ago, has shown these same kind of cracks, uniform in width from top to bottom. These walls were the foundation walls of a factory, carried up about 6 ft. above grade, and the superstructure was not placed on them for some time, so that they were not subjected to loading at all. The cracks appeared at remarkably uniform intervals; I remember it—I think there were two cracks in the length of each wall of 250 ft.

Mr. Lindsley. MR. C. E. LINDSLEY.—I have been curing with steam and have been putting up quite a number of houses, especially foundations, and never have had trouble. The blocks were properly made and did not crack.

Mr. Wig. MR. WIG.—It is not impossible that the cracks were due to contraction or change in volume of the cement, but they might be laid to many causes. They might be due to atmospheric tempera-

ture changes or to a poor quality cement. Some cements will de- Mr. Wig.
crease in strength with age after a period of increasing strength. We know that by numerous tests. These cracks may be caused by impurities or peculiarities of the sand or aggregate, peculiar weather conditions when the concrete was made or while still green, settling of the foundation. Unless there is a very careful examination of all the conditions that would affect the problem it is impossible to place the responsibility.

In regard to the coefficient of expansion of concrete, the coefficients of expansion of all materials that enter into concrete are very nearly the same. Professor Norton states there is very little difference with temperature changes; taking very low temperatures and very high temperatures, the coefficient of expansion was very nearly the same. There will be a difference, however, in different mixtures due to the elasticity of the mixture, you might say. In a lean proportioned mixture the material is more elastic and will allow more expansion and contraction in the material itself, but a rich mixture will not allow this expansion and contraction. While the individual particles do expand just as much in one as in the other, there are the more voids in the leaner mixture. The coefficient of expansion is practically .000005.

Mr. Gill.—I think the question of settlement of foundation Mr. Gill.
may be eliminated, because I have had so many instances where it was not reasonable to attribute it to that cause; cases where there was no evidence of cracking in the foundation. Another thing is, my experience has covered a wide territory and a number of different conditions, different cements and different materials, and this peculiar phenomena occurs under those varying conditions. The cracks develop within a certain time, I do not know how long, but from two or three or four weeks to three months after the wall has been laid. There has been a peculiar relationship between the occurrence of those cracks and the use of green blocks; that is, using certain materials under certain conditions and using old blocks, the building has been perfect; where by using the same materials under otherwise like conditions, but using blocks green, those cracks have developed. By green I mean 8 or 10 days old; steam cured 36 hours and then 8 or 10 days old. Of course, they are sound, good, hard blocks, but they apparently have not attained

Mr. Gill. that final condition which they reach say after 6 weeks or 2 months or a year. I do not know what it is, but there is some element involved that has not been explained. My experience has covered quite a range of conditions, localities and seasons, and that trouble occurs in every case with the use of green blocks, and I have never known it with the use of well-seasoned blocks, that is, blocks say 6 months old.

The President. PRESIDENT HUMPHREY.—Referring to the comparison of brick and concrete walls there is one thing that should be borne in mind. The ordinary brick is about 2 by 8 in., and in a given area of wall there are a good many more pieces in a brick wall than there would be in a concrete block wall. There are undoubtedly a good many cracks in a brick wall. Brick walls for the most part are laid up with lime mortar, and the expansion or contraction in a wall of considerable length may be taken up by the bricks themselves and the joints with a crack so small that the eye would not ordinarily notice, because the joint of the mortar disguises it. Whereas in a block wall, where the blocks are of large size, those cracks become more pronounced.

The contraction and expansion of walls due to temperature is common to all materials. Even a stone wall has expansion and contraction. The walls of old buildings have been laid with random courses or rubble walls and you find cracks have developed.

Now there is a good deal to be said on the subject of proportions. A great many cement blocks are made with five or six parts of a badly graded material, perhaps made under conditions where there is a minimum amount of water, and the result is a dry, porous mass, lacking in strength, and it is not surprising that cracks will develop, because even in walls there are more or less transverse strains brought upon the base in that wall. The block itself cannot have the necessary strength until it can resist those loads and that crack.

Now the foundation, whether a wall be of brick or stone or cement blocks, must be sufficient so that there will be no settlement. Cracks may occur around the joints if the settlement is not great, but if it is great they occur right through the block, unless the base be strong enough to resist.

ANNUAL REPORT OF THE EXECUTIVE BOARD.

The Objects of the Association.—Again we emphasize, as we have done in the past, that the National Association of Cement Users is primarily an educational institution. It was never intended that it should be commercial, and although we believe it is necessary and desirable that there be an exhibition in connection with our Convention, the former is for instructive purposes, and to demonstrate the best methods pursued in the preparation and handling of cement and concrete.

The Association is, therefore, supported by the dues and such revenue as may be derived from the Exhibition held in connection with our convention, contributions, and in a very small degree from the sale of its Proceedings.

Meetings.—Since the last Convention, the Executive Board has held two meetings and the Executive Committee met six times. In addition many conferences were necessary to carry on the work of the Association. An abstract of the minutes of these meetings is appended hereto.

Membership.—The membership at the close of the last meeting was 863, since that time 216 members have been added, 10 members have resigned and 21 members have been dropped for non-payment of dues. At the present time the membership is 1,048.

Change of Fiscal Year.—The action of the last Convention changed the beginning of the fiscal year, January 1st, to July 1st. This action was found necessary in order to facilitate the proper analysis of the expenses of the Association, by avoiding all overlapping of the accounts of the Convention in the annual financial report. The Convention of the Association occurs during the winter months, and the expenses incident to the preparation for and during the Convention had been found to appear in two fiscal years when the fiscal year ended December 31.

The Executive Board, therefore, in following out the action of the Association, sent out on July 1 the bills for dues for the ensuing year. The responses indicate the hearty approval of the members of the Association, as they have responded promptly and the change has now been effected.

Annual Statement of the Treasurer.

RECEIPTS.

Balance, January 1, 1910		$1,288.34
Dues	$2,949.99	
Dues, Contributing Members	425.00	
Publications, Sale	818.10	
Exhibition, Chicago	2,000.00	
Exhibition, Cleveland	1.50	
Membership Certificate	28.00	
Interest	9.98	
Contributions	850.00	
Convention	18.45	
Miscellaneous	6.22	
Total Receipts		$7,107.24
		$8,395.58

EXPENDITURES.

Salaries	$1,201.10	
Office Rent	225.00	
Office Equipment	34.50	
Postage, Printing, Stationery, etc.	$301.42	
August to December, 1909	213.28	
	514.70	
Sectional Committee Expense	35.35	
Meetings, Executive Board	55.05	
Publications, Including Volume V, 1909	4,355.93	
Convention, Chicago	797.44	
Exhibition, Chicago	33.60	
Membership Campaign	302.93	
Membership Certificate	7.50	
Membership Imprint	10.00	
Miscellaneous	27.81	
Total Expenditures	$7,600.91	
Balance, June 30, 1910	794.67	
		$8,395.58

PHILADELPHIA, December 8, 1910.

We hereby certify that we have audited the accounts of the Treasurer of the National Association of Cement Users for the six months ended June 30, 1910, and found that the dues collected and receipts from sales of publications, etc., as set forth in the cash book, had been properly accounted for, and that all disbursements were substantiated by vouchers.

The foregoing report of the Treasurer is in agreement with the books of account.

LYBRAND, ROSS BROS. AND MONTGOMERY,
Certified Public Accountants.

Proceedings.—The Proceedings of this Association continue to increase in size and in the character of the matter presented, until now they have become a valuable reference on matters pertaining to the proper use of cement. Purchase by technical societies and libraries, as well as by the members, indicate an increasing appreciation of their value. The Proceedings have always been an asset of the Association, although their cost has heretofore exceeded the immediate revenue derived from advertising and orders on hand. The sale from stock during the subsequent years has gradually made them a source of profit. This year, however, the sales of Proceedings and the advertisements have been more than sufficient to defray the cost of publishing, and the future sales will be an immediate source of revenue.

It is the aim of the Association to select with the greatest care the authors of the papers to be presented to the Annual Conventions, so that they may supply such information as to make the volume of Proceedings an authoritative book of reference. These Proceedings cover papers, and practical discussions of these papers, on the various uses of cement.

Standard Specifications.—The constantly increasing recognition of our Standards throughout this country and in Europe should be a matter of gratification to the members, and it is hoped that the members may co-operate in the campaign to secure the general adoption of these specifications. The development of the last Convention has indicated the desirability of preparing recommendations for standard practice which would cover methods in detail to be followed with rules governing the quality of the product and test requirements. This will simplify the work of the various committees since many of them found it extremely difficult to present a specification without some qualification, and this has burdened the specification with matter not strictly relevant to it. By means of this sub-division the committees can present a recommended practice governing the preparation of the product covered by the specifications.

Convention.—The Executive Board has found it desirable to continue its affiliation with the Cement Products Exhibition Company under satisfactory financial relations. This relieves the Association of an enormous amount of work. These annual cement

exhibitions have so developed in character and size as to make them an undertaking of considerable magnitude, and requiring months of systematic arrangement. It seems to the Executive Board that this can be better handled by an Exhibition Company as a commercial proposition, at least for the present.

Headquarters.—As the membership of the Association increases, the amount of detail work incident to carrying on the daily work of the Association necessitates an increase in the clerical force and additional space for offices in which this work may be performed.

The work incident to editing our growing Proceedings, the issuance of bulletins in addition to the current correspondence with the members necessitates a proper business organization, that it may be properly handled and with facility. The expense incident to this, we believe, is considerably offset by the improved financial condition of the Association, the increase in revenue derived from the sale of publications and the number of members.

Income.—At no time since the Headquarters were instituted under the instructions of the Convention has the money derived from contributing memberships been anything near the amount which was promised by the special committee, as a guarantee for the increased expenses incident to the establishment of these headquarters. It has been necessary for the President and other members of the Executive Board to secure contributions of large size from interested corporations, which have greatly added to the revenue which is now derived from the membership on the basis of the very low annual dues of $5 per member.

The number of members is gradually reaching the point, which is desired by the Executive Board, where the dues will be sufficient for the expenses of the Association. This seems to be the end which should be sought, because after all this is the safest basis for the operation of the Association. It should not be necessary for the members of the Board to be obliged to solicit funds in order to avoid deficits in the finances.

Inasmuch, as the finances of the Association have been brought down by economical methods to the lowest possible point, it does not seem possible to effect any saving in this direction. The annual increase in the expense of the Association is incident to the in-

creased growth and activity. The members of the Association have responded in a measure to the appeal of the Executive Board for increased membership, but it is evident that with our present membership, if each member would secure an additional member, the total would reach the point where the Association would be self-supporting.

It is the opinion of the Executive Board, based on a study of the finances of the Association, that the deficit existing on January 1, 1910, will be entirely wiped out by the end of the present fiscal year.

The Executive Board would recommend that the Association pass a resolution expressing its appreciation and thanks to those who have given financial aid by special contributions or as contributing members.

Our President.—In conclusion the Executive Board is pleased to record its appreciation of the continued interest and active work given the Association by your President, Mr. Richard L. Humphrey.

The excellence of the present program and the wide scope of the papers contributed has been the result of the President's own effort, made possible alone by his wide acquaintance and active participation in the technical subjects of special interest to this Association. Attention is also called to the continued improvement in the appearance and arrangement of the Proceedings, and in this is also seen the guiding hand and earnest work of your President.

Respectfully submitted on behalf of the Executive Board,

E. D. Boyer,
M. S. Daniels,
L. C. Wason,
Committee to Prepare Annual Report.

APPENDIX.—MINUTES OF MEETINGS OF THE EXECUTIVE BOARD.

Meeting of the Elective Members of the Executive Board, Held March 9, 1910, at 11 Broadway, New York, N. Y.

Present: Richard L. Humphrey, President; Edward D. Boyer, First Vice-President; M. S. Daniels, Second Vice-President; and E. S. Larned, Third Vice-President.

On motion, Edward D. Boyer and M. S. Daniels were elected members of the Executive Committee of the Executive Board.

On motion it was unanimously agreed that the Assistant to the President, Edw. E. Krauss, be directed to perform the duties of the Secretary until such officer may be elected by the Executive Board, and that all vouchers be endorsed by him as "Acting Secretary."

The committee also discussed the suggestion of the President that the Association issue a bulletin, perhaps quarterly.

The question of membership was thoroughly discussed, and it was agreed that the work of securing new members should be done through the office of the Association, and that it would be desirable to have all letters and other circular matter, to be used by the Membership Committee, prepared in the office of the Association and sent to the various members of this committee.

The bill of the John C. Winston Company, amounting to $3,718.79 for printing, was considered, and the President authorized to have the same paid.

Inasmuch as it was deemed inexpedient, in view of the present condition of the finances of the Association, to call a meeting of the Executive Board, it was decided to send out the minutes of the meeting of the Elective Members for the purpose of securing the ratification of the entire Board. The members of the Executive Board at the same time to be invited to send in any suggestions or discussions relative to the work of the Association.

Executive Committee Meeting, Held June 11, 1910, at 11 Broadway, New York, N. Y.

Present: Richard L. Humphrey, President; Edward D. Boyer, M. S. Daniels, and H. C. Turner, members of the Executive Committee.

The report of the Auditors was presented and the finances of the Association discussed. It was decided to send out the bills for dues for the fiscal year beginning July 1, 1910, accompanied by a letter

explaining that the $5.00 dues covered one fiscal year, and that while the date of payment had been advanced, the members would receive the same return for their money as in previous years.

The question of advertising in the Proceedings was discussed, and it was decided to delay the publication of the Proceedings until sufficient money was in sight to warrant the expense.

The campaign for Contributing Membership was also discussed and means devised for making it more effective.

The committee then considered the proposition of the Cement Products Exhibition Company to hold the Seventh Annual Convention and Exhibition of this Association conjointly with its exhibition in New York. The Executive Committee accepted the proposition of the Cement Products Exhibition Company and decided to hold the Seventh Annual Convention and Exhibition in Madison Square Garden, December 12-17, 1910, dates of the Exhibition being December 14-21, inclusive.

The committee then adjourned to visit the Madison Square Garden in order to settle a number of details in connection with the Convention.

BOARD MEETING, HELD OCTOBER 12, 1910, AT THE HOUSE OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS, NEW YORK, N. Y.

Present: Richard L. Humphrey, President; Edward D. Boyer, C. W. Boynton, M. S. Daniels, P. S. Hudson, E. S. Larned, L. C. Wason, Vice-Presidents, and H. C. Turner, Treasurer.

Mr. Edward Boyer, was appointed Secretary, *pro tem.*

The minutes of the meetings of the elective members of the Executive Board, held February 25 and March 9, 1910, and of the Executive Committte, held June 11, 1910, were approved as read.

Mr. H. C. Turner, Treasurer, presented a financial statement which was discussed.

On motion, the action of the Treasurer in circularizing architects and others within a radius of 100 miles of New York City, regarding advantages of membership in the Association, was approved.

The action of the Executive Committee in guaranteeing the Trunk Line Association that 1,000 railroad certificates would be presented at the New York Convention so as to enable the viseing of certificates as presented, guaranteeing a return at one-half fare, in view of the fact that the Cement Products Exhibition Company has guaranteed the Association against loss, was approved.

The report of the tellers of the letter ballot, canvassed April 25, 1910, as reported in Circular No. 24, August, 1910, was, on motion, accepted.

The President was instructed to appoint a committee of three to prepare the Annual report of the Executive Board, which report is to be presented to the Executive Board for approval.

The request of the Concrete Products Manufacturers' Association of New Jersey that they be allowed to present the object of their organization at the Seventh Annual Convention was granted.

The program and banquet of the Seventh Annual Convention were discussed, and the President was authorized to appoint a committee to have charge of the same.

On motion it was decided that invitations for future Conventions be read at the business session by title only.

Adjournment.

BOARD MEETINGS, HELD DECEMBER 12-17, 1910, AT THE PRINCE GEORGE HOTEL, NEW YORK, N. Y.

Present: Richard L. Humphrey, President; Edward D. Boyer, M. S. Daniels, E. S. Larned, A. E. Lindau, L. C. Wason, Vice-Presidents, and H. C. Turner, Treasurer.

The minutes of the meeting of the Board, held on October 12, 1910, were read and approved.

The Executive Committee of the New York Local Committee met with the Board, and the detailed arrangements for the Convention were discussed as well as the banquet, to be held on Thursday evening.

On motion it was decided to order 1,500 guest badges as well as badges for the Local Committee.

On motion, the President was authorized to have 2,500 additional programs without cover printed, for the purpose of mailing immediately to the engineers and architects in the vicinity of New York, under the direction of the Local Committee.

The Treasurer presented his Annual Report, which was accepted.

On motion of Mr. Daniels, an Auditing Committee, consisting of Messrs. Larned and Boyer, was appointed to examine and audit the accounts of the Treasurer.

On motion, in accordance with the by-laws, a Nominating Committee was appointed, consisting of:

Thomas M. Vinton, *Chairman*, New York, N. Y.
Emile G. Perrot, Philadelphia, Pa.
Robert A. Cummings, Pittsburgh, Pa.
Robert Anderson, Cincinnati, Ohio.
A. T. Bradley, Rochester, N. Y.

On motion, a Committee on Resolutions was appointed, consisting of:

B. F. Affleck, *Chairman*, Chicago, Ill.
A. S. Johnson, St. Louis, Mo.
Peter Gillespie, Toronto, Canada.
Angus B. MacMillan, Boston, Mass.
Milton Dana Morrill, Washington, D. C.

Moved by Mr. Boyer and seconded by Mr. Daniels that the applications for membership received to date be approved, and that they become members immediately upon payment of dues.

The Executive Board agreed to appoint the following Sectional Committees for the ensuing year:

Building Blocks and Cement Products.
Exhibition.
Fireproofing.
Insurance.
Reinforced Concrete and Building Laws.
Roadways, Sidewalks and Floors.
Treatment of Concrete Surfaces.

The Annual Report of the Executive Board was presented and approved.

The following amendment to the by-laws was approved:

Amend Article I, Section 4, so that the second sentence reads as follows:

The Secretary shall submit monthly, or oftener if necessary, to each member of the Executive Board for letter ballot a list of all applicants for membership on hand at that time with a statement of their qualifications, and a two-thirds majority of the members of the Executive Board shall be necessary to an election.

Amend Article I, Section 4, by adding the following paragraph:

Applicants for membership shall be qualified upon notification of election by the Secretary by the payment of the annual dues, and unless these dues are paid within 60 days thereafter the election shall become void. An extract of the by-laws relating to dues shall accompany the notice of election.

MEETING OF THE ELECTIVE MEMBERS OF THE EXECUTIVE BOARD, HELD DECEMBER 19, 1910, PRINCE GEORGE HOTEL, NEW YORK, N. Y.

Present: Richard L. Humphrey, President; Edward D. Boyer, E. S. Larned, and Ira H. Woolson, Vice-Presidents.

On motion, Mr. H. C. Turner was elected Treasurer and Mr. Edw. E. Krauss, Secretary for the ensuing year.

On motion, Chairmen of the Sectional Committees were appointed as follows:

Building Blocks and Cement Products—Percy S. Hudson.
Exhibition—H. S. Doyle.
Fireproofing—Rudolph P. Miller.
Insurance—William H. Ham.
Reinforced Concrete and Building Laws—A. E. Lindau.
Roadways, Sidewalks and Floors—C. W. Boynton.
Treatment of Concrete Surfaces—Leonard C. Wason.
Adjournment.

REGISTER OF ATTENDANCE.
SEVENTH CONVENTION.

Richard L. Humphrey, Philadelphia, Pa.
Edward D. Boyer, New York, N. Y.
M. S. Daniels, New York, N. Y.
E. S. Larned, Boston, Mass.
Henry C. Turner, New York, N. Y.
Edward Eugen Krauss, Philadelphia, Pa.
C. W. Boynton, Chicago, Ill.
Rudolph P. Miller, New York, N. Y.
Alfred E. Lindau, St. Louis, Mo.
W. H. Ham, Boston, Mass.
Leonard C. Wason, Boston, Mass.

Abbey-Dodge-Brooks Concrete Company, The. Raymond E. Brooks, Newark, N. J.
Aberthaw Construction Company. L. C. Wason, Boston, Mass.
Affleck, B. F., Chicago, Ill.
Aldridge, E. V., Pittsburg, Pa.
Allentown Portland Cement Company. I. Murray, Allentown, Pa.
Alpha Portland Cement Company, Easton, Pa.
American Cement Company. C. M. Camm, Philadelphia, Pa.
American Concrete Steel Company. John T. Simpson, Newark, N. J.
American Silk Dyeing and Finishing Company. Thomas J. Arnold, Hawthorne, N. J.
American Steel and Wire Company. H. S. Doyle, Chicago, Ill.
Anderson, Robert, Cincinnati, Ohio.
Andrews and Company, Incorporated, F. M. David Gutman, New York, N. Y.
Armstrong Cement Works. P. H. Atwood, Armstrong, Iowa.
Arp, Clarence K., Chicago, Ill.
Ash Grove Lime and Portland Cement Company. W. B. Phillips, Kansas City, Mo.
Aspin, John D., Malden, Mass.

Atlas Portland Cement Company. John R. Morron, New York, N. Y.
Auerbach, Henry, New York, N. Y.
Ballinger, Walter F., Philadelphia, Pa.
Ballou Company, F. M. F. B. Follett, Providence, R. I.
Balyeat, R. M., Van Wert, Ohio.
Bauer, Harry E., Philadelphia, Pa.
Beach, Frank R., Irvington, N. J.
Beck, A. L., Ada, Okla.
Beck, J. P., Chicago, Ill.
Benson, Newton D., Providence, R. I.
Berry, Herman Claude, Philadelphia, Pa.
Bickelhaupt, F. R., Cleveland, Ohio.
Birnie Company, Alexander C. A. C. Birnie, Ludlow, Mass.
Blanc Stainless Cement Company. J. Maxwell Carrere, Allentown, Pa.
Blethyn, Benjamin, Washington, D. C.
Boorman, T. Hugh, New York, N. Y.
Bouton, Frank, Rochester, N. Y.
Boyd, Robert Wright, New York, N. Y.
Boylan, William H., New Brunswick, N. J.
Boyle, James, Freeland, Pa.
Brett, Allen, New York, N. Y.
Brown, Burtis S., Boston, Mass.
Brown, Charles C., Ebenezer, N. Y.
Brown Hoisting Machinery Company. George T. Links, Cleveland, Ohio.
Brown, James Henry, Brooklyn, N. Y.
Bruff, James L., Buffalo, N. Y.
Budd, John W., Des Moines, Iowa.
Buser Concrete Construction Company. W. E. Buser, Mount Morris, Ill.
Bush, R. G., Plainfield, N. J.
Canadian Cement and Concrete Association, Toronto, Canada.
Carmichael Company, William P. William P. Carmichael, St. Louis, Mo.
Case, C. J., Troy, Pa.
Cathcart, Robert, Cleveland, Ohio.

Cement. R. B. Sears, New York, N. Y.
Cement Age. E. A. Trego, Philadelphia, Pa.
Cement Era. E. S. Hanson, Chicago, Ill.
Cement World. E. B. Wolfram, Chicago, Ill.
Chain Belt Company, Milwaukee, Wis.
Chandler, Dana G., Sylvania, Ohio.
Chapman, Howard, New York, N. Y.
Chase, Charles F., New York, N. Y.
Cohen, A. B., Hoboken, N. J.
Collier, D. M., Albany, N. Y.
Commonwealth Roofing Company. Horace V. Smith, Newark, N. J.
Concrete. Walter C. Boynton, Detroit, Mich.
Concrete Age, The. George C. Walters, Atlanta, Ga.
Concrete Products Company, The. Ross F. Tucker, New York, N. Y.
Concrete Separator Company. W. B. Johnston, New York, N. Y.
Concrete Steel Company, New York, N. Y.
Concrete-Steel Engineering Company. William Mueser, New York, N. Y.
Concrete Stone and Sand Company, The. A. A. Pauly, Youngstown, Ohio.
Concrete Utilities Manufacturing Company. Stanley B. Kirk, Westbury, N. Y.
Conk, Harry M., Long Branch, N. J.
Conk, William, Long Branch, N. J.
Converse and Company, H. P. Edwin P. Bliss, Boston, Mass.
Cordes, Charles J. T., Waldwick, N. J.
Corrugated Bar Company. W. H. Burk, St. Louis, Mo.
Corrugated Bar Company. W. S. Thomson, Chicago, Ill.
Cox, Charles H., Philadelphia, Pa.
Crawford, Robert, Chicago, Ill.
Crossett and Lloyd Concrete Company. Frank Crossett, Binghamton, N. Y.
Crozier, Alfred O, Wilmington, Del.
Culyer, Thurston C., Purdy Station, N. Y.
Cummings Structural Concrete Company. Robert A. Cummings, Pittsburg, Pa.

Cunard-Lang Concrete Company, The. A. C. Uhle, Columbus, Ohio.
Cunningham, Jr., Stanley, New York, N. Y.
Cuozzo, Donato, New York, N. Y.
Cutler, Elihu H., Springfield, Mass.
D. and A. Post Mold Company, The. G. H. Dougherty, Three Rivers, Mich.
Dane, Nathan, New Haven, Conn.
Davis, Benjamin Herman, New York, N. Y.
Davis, Chandler, New York, N. Y.
Denman, Walter M., Springfield, Mass.
Densmore and LeClear. Gifford LeClear, Boston, Mass.
Denton and Company, Incorporated. P. E. Eisenmenger, New York, N. Y.
Deposit Coal and Cement Company. W. J. Putnam, Deposit, N. Y.
De Smet, George W., Chicago, Ill.
Dexter Portland Cement Company. Joseph Brobston, Nazareth, Pa.
Dietrichs Clamp Company. Charles Dietrichs, Little Ferry, N. J.
Dings, L. E., Sylvania, Ohio.
Dixon, De Forest H., Brooklyn, N. Y.
Douglas, Walter J., New York, N. Y.
Drake, James B., Oneida, N. Y.
Duboyce, William J., Philadelphia, Pa.
Dumary, T. Henry, Albany, N. Y.
Dyer, Wesley E., Muskegon, Mich.
Eastlock, J. H., Swedesboro, N. J.
Edison Portland Cement Company, The. W. S. Mallory, Stewartsville, N. J.
Eilbacher, Joseph F., Elizabeth, N. J.
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GUESTS.

Banneth, M., Special Representative of the Lord Mayor, Budapest, Hungary.

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Bitner, Karl, Director, Oesterreichischer Betonverein, Vienna, Austria.

Black, Colonel William M., U. S. A., Corps of Engineers, New York, N. Y.

Boehm, Franz, Engineer, Oesterreichischer Betonverein, Vienna, Austria.

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Russell, W. A., Pittsburg, Pa.

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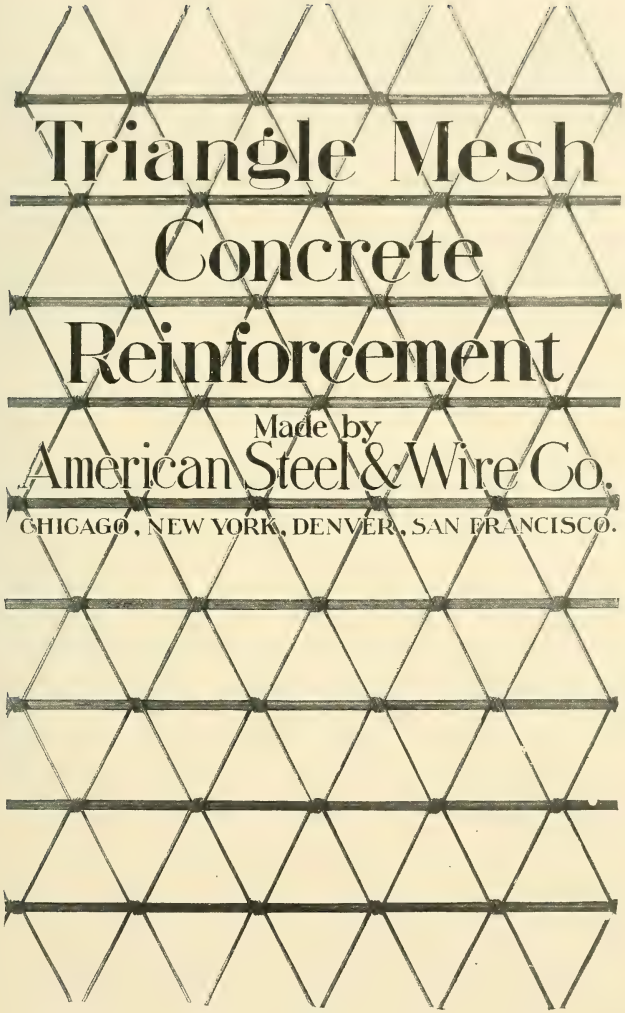
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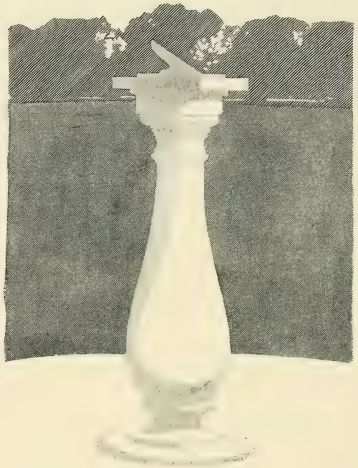

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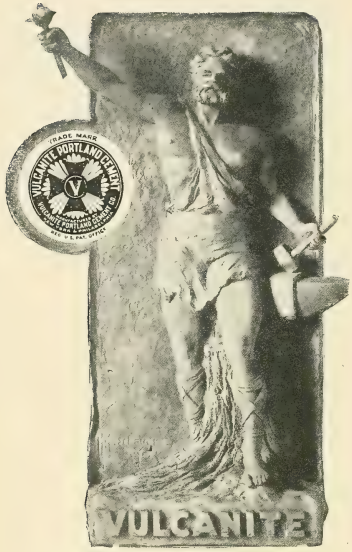

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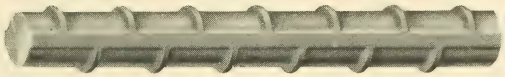

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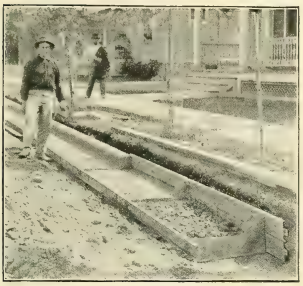

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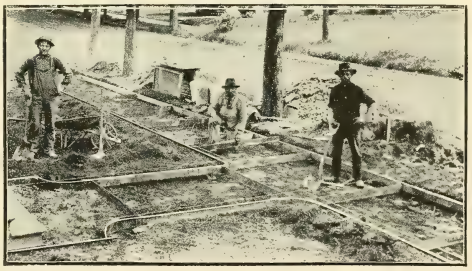

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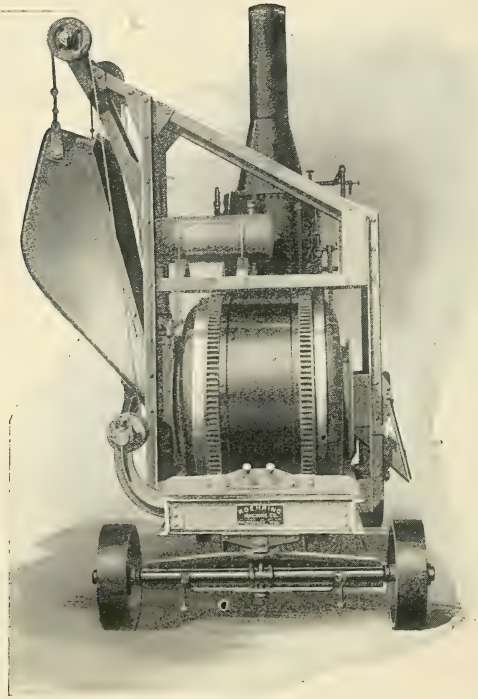

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CONTRACTOR.—But they said that their deformed bar had no excess weight; that it would weigh the same as a plain bar of the same size.

ENGINEER.—How could it weigh the same? If there was one square inch of area between those cross-ridges, then we could have planed off those ridges and had a bar the same area and weight as a plain bar. When we bought their deformed bar, we bought a plain bar plus those ridges, and the excess weight which is now shown on our bills is the weight of those same cross-ridges for which we have to pay, but for which we receive no credit from the Owner's Engineers.

CONTRACTOR.—That's all very fine, but we *had* to have a bar with a mechanical bond. We would have been in the same fix if we had bought the Havemeyer bar, which you recommended. Look at the ridges on that!

ENGINEER.—That's just my point. We wouldn't have had this excess weight if we had bought the Havemeyer bar. That is the best rolled deformed bar on the market, because it has the same weight and net area of cross section as a plain bar. The metal that forms the ridges on the face of the bar is pressed out of the edge. If we had bought Havemeyer bars our bill for reinforcement would have been the same as my estimate.

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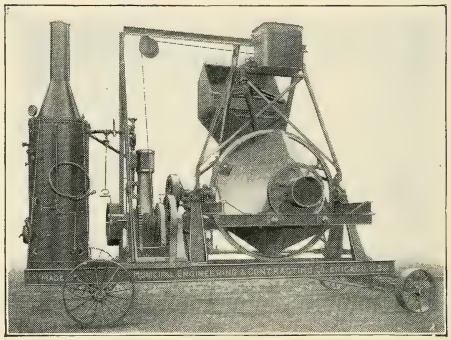

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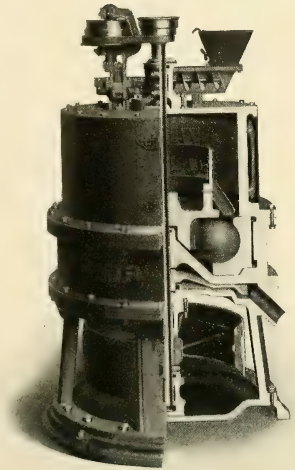

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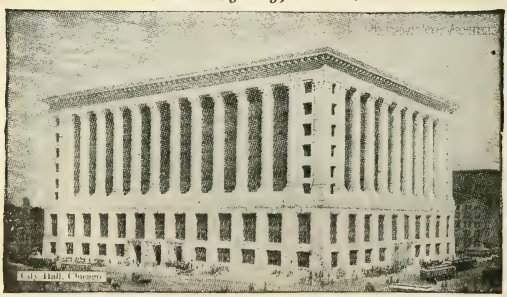

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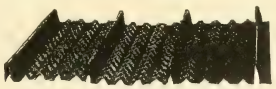

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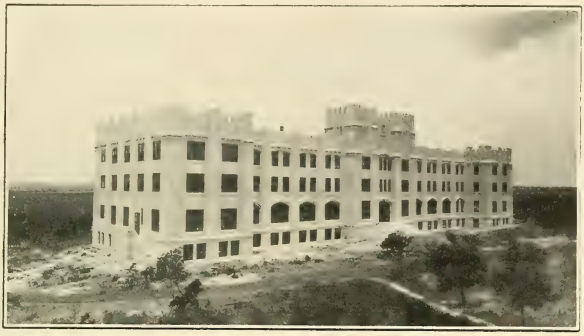

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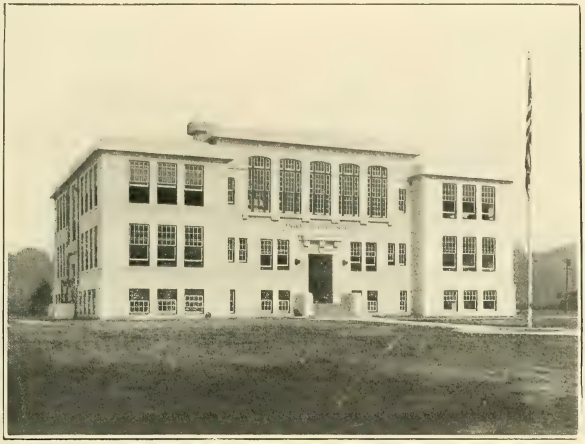

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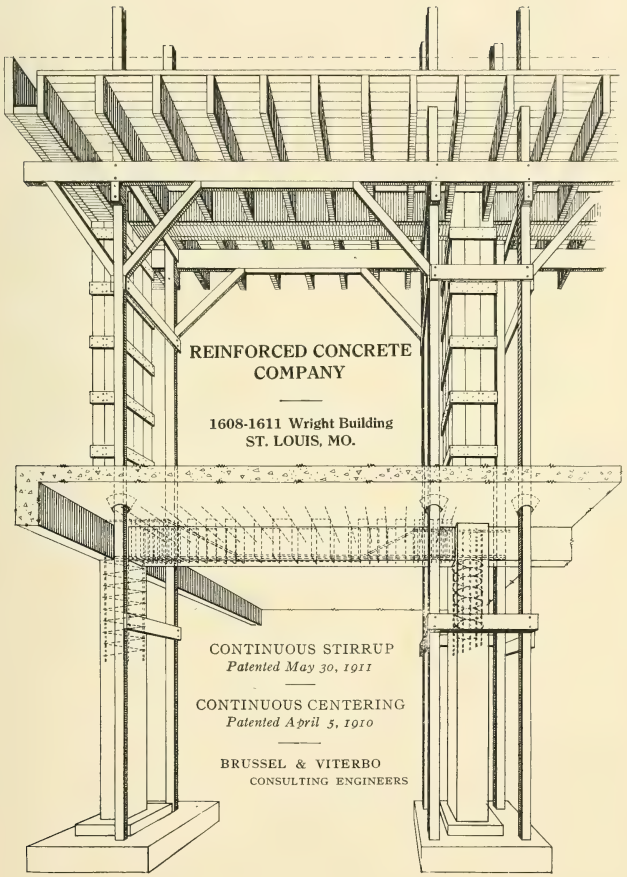
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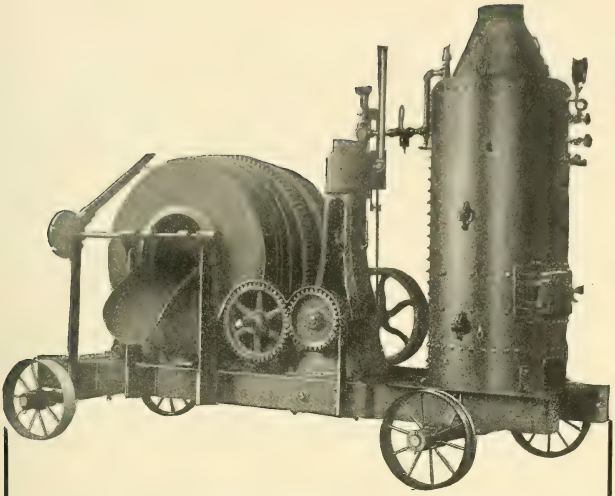

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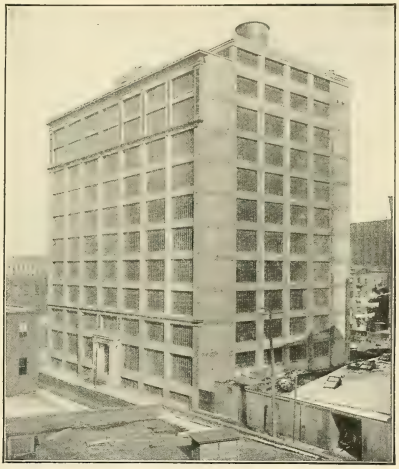

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